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Effect of Increasing Truck Weight on Bridges

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EFFECT OF INCREASING TRUCK WEIGHT ON BRIDGES

by

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MASTER OF SCIENCE IN CIVIL ENGINEERING

ABSTRACT

The issue of increasing the gross vehicle weight limit on the interstate highway system has been debated on the local and federal levels. The driving forces for and against this topic cover a broad spectra ranging from economic benefits to public safety. The University Transportation Center of Alabama is sponsoring this study assessing the force effect that bridges experience when travelled by vehicles with a 97,000-lb gross vehicle weight (GVW). The maximum internal shear and moment caused by two proposed trucks (97-S and 97-TRB) are compared to effects from three base models: design live loadings from the *American Association of State Highway and Transportation Officials* (AASHTO) *Standard Specifications* and *AASHTO Load and Resistance Factor Design* (LRFD) *Specifications*, and the envelope from five potentially critical Alabama legal loads. Hypothetical simple span bridges and two-span continuous bridges with a 1:1 span ratio are analyzed with each load model, providing data that correlates to the impact that increased truck weight has on bridges. Results show that the shorter 97-S causes greater shear and moment compared to the 97-TRB on all simple spans and a large percentage of the continuous span bridges investigated. The design live loading issued in the AASHTO Standard Specifications does not generate adequate force effects that fully envelope the effects from 97-kip vehicles. Depending on span length and bridge type, both proposed models will exhibit force effects above those from the envelope of five Alabama legal loads. However, effects initiated by the LRFD model envelope all shear and moment effects of both 97-kip trucks on each of the two bridge types. It is concluded that LRFD methods represent significant benefits to bridge design practices concerning the implementation of heavier trucks on the interstate highway system.

Additionally, the deck reinforcement specified by the *Alabama Department of Transportation* (ALDOT) *Bridge Bureau's* standard slab detail is checked using the LRFD Specifications and the 51-kip tri-axle load of the 97-kip vehicles. For several girder spacings, it is determined that the primary reinforcement utilized in ALDOT's standard slab design does not meet the strength requirements of AASHTO LRFD.

Keywords: truck weight increase, bridge design, LRFD Specifications, bridge live load

DEDICATION

I would like to dedicate this thesis to my parents, David and Ashley Yates. Through their unconditional love and support, they have instilled the core foundation of my intellectual aspirations. I will forever be grateful.

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CHAPTER 1 INTRODUCTION AND LITERATURE REVIEW

1.1 Introduction

The US House of Representatives has proposed legislation (H.R. 763 "Safe and Efficient Transportation Act of 2011") allowing states to authorize the use of vehicles with a gross weight of 97,000 pounds on the Interstate Highway System if: (1) the vehicle has a minimum of six axles, (2) single axles do not exceed 20,000 pounds, (3) tandem axles do not exceed 34,000 pounds, (4) any grouping of three or more axles does not exceed 51,000 pounds. The general intent of this legislation is to promote economical prosperity and uniformity among US states and bordering nations as described in the North American Free Trade Agreement established in the mid 1990's.

The main objective of this thesis entails analyzing the critical force effects of simply supported and two-span continuous bridges under two truck configurations that represent the criteria of the proposed legislation. Using engineering principles along with structural analysis software, the critical shear and moment effects from these heavier trucks will be compared to the effects generated by three base models: AASHTO Standard Specifications, AASHTO LRFD Specifications, and five Alabama Legal Loads. Maximum shear and moment ratios provide the necessary data for quantifying the effects so reasonable assessments can be made about bridge infrastructure.

Also included is a comparative analysis of the standard bridge slab design issued by Alabama Department of Transportation. The reinforcement provided by the standard slab chart will be investigated using Load and Resistance Factor Design methods and the critical axle grouping of the 97,000 pound vehicles. The results from this sensitivity analysis will aid Alabama and other state departments of transportation with a few preliminary steps along the inevitable path to heavier trucks on the Interstate Highway System.

1.2 Reasons for Gross Vehicle Weight Policy Change

Economic projections indicate that freight commodities are rapidly on the rise. In the United States, 12.8 billion tons of freight was transported by truck in 2007. Due to lingering recession impacts, only 10.9 billion tons were moved in 2009 but 18.4 billion tons are expected in 2040, an increase of over 68%. Without any expansion to the national highway system, roadway segments already experiencing congestion are assumed to increase by nearly 400% between 2007 and 2040 (USDOT Freight Facts 2010). In Alabama, this heavy truck traffic will directly affect segments of I-59/I-20, I-65, and I-10 around the Birmingham, Montgomery, and Mobile areas respectively. Non-interstate highways expecting increased congestion include US 431 and US 280 (ALDOT Freight Study, 2010). As the economy rises, diesel fuel prices are expected to increase which will raise operating costs of transports. The "trickle-down effect" will take place as commodity prices will increase if truck size and weight limits are not reformed.

Along with easing congestion, an increase in gross vehicle weight (GVW) will help provide uniformity with neighboring countries Canada and Mexico. In part, the North American Free Trade Agreement (NAFTA) of 1994 was established for this reason, but the varying truck size and weight standards of each country confine the effectiveness of this agreement. Mexico has a maximum GVW limit around 107-kip while some provinces in Canada are operating near 129-kip depending on axle spacing. The US has the lowest maximum limit of 80-kip. Special NAFTA permits are issued for overweight loads, but this process restricts the overall efficiency of the import/export trade scenarios (TRB 1990).

1.3 Impacts of Increasing Truck Weight

There are a multitude of impacts that increasing truck weight will have on trucking industries as well as the tangible impacts felt by others. Several key effects include, but are not limited to economic productivity, environmental, safety, and highway infrastructure costs. Whether these impacts are considered beneficial or disruptive often depend on perspective.

1.3.1 Economic Productivity

The economic productivity deriving from increased GVW is a relative issue benefitting some while hindering others. Agencies that currently transport bulk commodities at a GVW around 80-kip will benefit from weight increases as their payload subsequently rises. This will reduce operating cost on a per trip basis. Due to the competitive nature of the shipping business, this carrier operating cost savings would no doubt trickle down to the freight distributors because a reduction in vehicle miles of travel will be provided. A study done in the 1980's concluded that annual savings of \$3.2 billion would result if the proposed 9-axle (one single and four tandem axles) Turner Double with a GVW of 105-kip became legal (Figure 1.1). Based on historical freight data it was estimated that one-fourth of the total miles traveled by combination trucks would take place in Turner Doubles. From another angle, increased truck weight and lower shipping cost will reduce the volume of freight transported by rail as current manufactures utilizing the railroad system will have cost incentives to make the switch to truck carriers (Cohen, Godwin, Morris, and Skinner 1987).

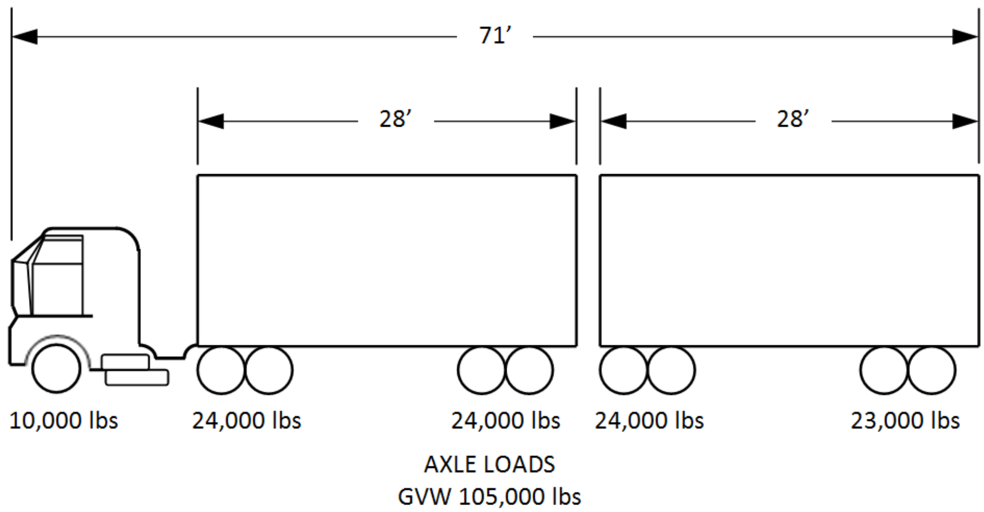


Figure 1.1 – Turner Double

1.3.2 Environmental

Fuel consumption, on a freight ton hauled per gallon burned basis, will decrease if larger loads are permitted. Hauling an abundance of commodities from an arbitrary origin A to location B will reduce the total number of trips required hence limiting the number of vehicle miles of travel (VMT) and the fuel consumed. However, the added freight to truck transport switching from rail will increase annual gross fuel consumption. Comparative information on train versus truck emissions and efficiency was not investigated. A slight drawback from increasing truck weight limits is the increased noise. Truck noise is a function of engine type, speed, and tire properties. No recent history on noise studies between truck types was discovered but it is rational to assume increased GVW will increase engine strain hence the noise level. As property value is affected by noise, it is predicted that noise will have an impact, but the degree of the impact is not apparent (USDOT TS&W Vol-I 2000).

1.3.3 Safety

Safety becomes a major concern when considering changes to truck size and weight. The majority of the general public included in focus groups pertaining to weight regulations expressed negative concerns with allowing heavier trucks on roadways (USDOT TS&W Vol-I 2000). However, crash rates from LCV's closely resemble those of five-axle semi-trailers with GVW under 80-kip but data is not always available since truck length and weight are not usually included in accident reports. For vehicles with additional axles above that of the standard five-axle semi-trailer, braking capacity will be enhanced due to advanced technology in the motor vehicle industry. Each additional axle can be equipped with braking mechanisms to help combat against the increased momentum that heavier trucks demonstrate.

One factor directly related to safety that can be measured is the vehicles' stability and control. Vehicle rollover is a leading concern to safety when discussing the allowance of heavier trucks on the National Network (NN). Rollover is a function of speed, GVW, axle length, suspension type, and tire properties. It occurs in two basic scenarios. The first is caused by high speeds when negotiating a steady-state turn. Every vehicle has a static roll stability (SRS) threshold which decreases with an increasing center-of-gravity. If the SRS value is exceeded, the vehicle will overturn. The second rollover scenario entails high speeds where evasive maneuvers have taken place much like the phenomena of cracking a whip. Factors that play a key role in these situations involve the number of articulation points and the dynamic roll stability (DRS) of each vehicle. Semi-trailers have one articulation point while double and triple trailer combinations usually have three and five points respectively. Susceptibility to rollover magnifies with the addition of articulation points as the DRS is lowered.

In order to sustain safety, several issues need to be addressed. It has been recommended that operators of heavier motor vehicles extend their training with certified programs and receive monetary incentives to ensure operations are carried out at superior safety levels. Subpar roadway conditions and geometrics should be rehabilitated as well as dated equipment that do not meet all safety criteria (USDOT TS&W Vol-I 2000).

1.3.4 Highway Infrastructure Costs

An increase in GVW will have substantial effects on highway infrastructure with roadway and bridge improvement costs. In past circumstances observed, specifically focusing on modifications to vehicle configurations, annual repair costs to roadways remain quite stationary if not being reduced. Since the federal government has capped single axle (20-kip) and tandem axle (34-kip) weight limits, innovative configurations maintain this limit and frequently suggest slightly lowering it. Pavement wear is directly related to individual axle loadings rather than gross vehicle weight. Referring to the study involving the Turner-Double, it was determined that a 50% reduction in equivalent single axle loads (ESALs) will result when compared to the standard five-axle 80-kip semitrailer. This would prevent 15-billion ESAL miles per year. At the time of this study, pavement repair costs averaged 1.6 cents per ESAL mile producing a cumulative annual savings of $250 million for state departments of transportation (DOT) (Cohen, Godwin, Morris, and Skinner 1987).

The Comprehensive Truck Size &Weight Study of 2000 sponsored by the USDOT compared two semi-tractor trailers both with a 12-kip load on the steering axle. A five-axle truck had two tandem axles of 34-kip with a GVW of 80-kip. The second truck was configured with six-axles including one tandem axle with the same axle weight as the first vehicle but a rear tri-axle axle of 44-kip having a GVW of 90-kips. According to their estimate in regards to flexible pavement surfaces, the five-axle truck will cause 18% more roadway damage per VMT than the six-axle combination, despite having a 11% reduction in gross weight (USDOT TS&W Study Vol-II 2000).

On the other hand, state DOTs will see an increase in the funding required for bridge rehabilitation if GVW limits are increased. Previous studies conducted by the United States Department of Transportation (USDOT) Federal Highway Administration (FHWA), Transportation Research Board (TRB), and others have determined that repair cost from bridge damage will be the greatest single highway infrastructure cost due to heavier trucks. Estimating the net cost for bridge repair is a detailed and complicated process because a degree of uncertainty is always present. It requires the composite sum of several cost factors only reasonably estimated at a global level. These main factors can be summarized as: cost of construction, cost due to diminished service life, and user costs.

Construction costs include the price of building new bridges and/or rehabilitation to those existing. Reducing the service life of a bridge adds additional costs that are not accounted for during the design phase. Every interstate bridge is designed with a service life under a notional design loading. Allowing applied loads above that of the design load negatively affects its service life expectancy. Construction costs are directly representative of the increased shear, moment, and fatigue effects felt by bridge elements due to these increased loadings.

Short-term effects result from overstressing bridge elements. Overstressing a bridge causes cracks in its girders and deck, diminishing the load-carrying capacity and eventually resulting in closure or failure. Once signs of overstressing are apparent, the bridge owner has three options: replace the bridge, strengthen the bridge, or post weight limits. Bridge type typically governs the capability of being strengthened. Studies show that the cost of strengthening reinforced concrete (RC) bridges and prestressed bridges can equal the cost of replacing them.

Long-term effects of overstressing are seen in gradual fatigue damage. After numerous loading cycles, bridges show signs of fatigue witnessed by the cracking of the superstructure at locations of high stress. Fatigue directly reflects a shorter life span of a bridge and the cost effects of fatigue are entangled in its reduced life. Steel bridges are at a greater risk of experiencing fatigue but studies show that prestressed concrete bridges and RC decks can exhibit fatigue symptoms if continually overloaded (TRB 1990 and Weissmann and Harrison 1998).

Increased user cost is a result felt by the daily traffic. Essentially it is a function of time delay caused by bridge repair. During this time bridges will either be closed and the traffic rerouted or partially closed causing traffic to merge into single lanes. In any case traffic flow will be affected. A tangential part of user costs is also found in additional vehicle maintenance and fuel consumption stemming from rerouting and traffic congestion (Weissmann and Harrison 1998).

1.4 History of Truck Size & Weight Regulations

In the early 1900s truck size and weight limitations were governed on a per state basis with the focal point of protecting state highways and bridges. However, only a small percentage of states adopted any regulations at all. In 1932 the American Association of State Highway Officials (AASHO) suggested guidelines for single and tandem axle weight limits and by 1933 all states had truck size and weight regulations of some kind. The AASHO policy of 1946 reformed that advised in 1932 and proposed that state agencies limit single axles to 18-kip and tandem axles to 32-kip. A maximum gross vehicle weight of 73.28-kip was also suggested for "vehicles having a maximum length of 57-ft between the extremes of the axles" (TXDOT 2009). This was the first instance that related the notion of GVW to axle spacing. The contents of the Federal-Aid Highway Act of 1956 established that all interstate highway improvements were to be funded with a 90/10 split between federal and state governments respectively. Due to the sizeable investment from the federal government, the regulation recommendations by AASHO in 1946 became federal policy. If states accepted higher weights prior to the adoption of the 1956 Act, they were allowed to continue to operate under a "grandfather clause". Vehicle width was also set to a maximum of 96-in. but height and length restrictions were still left to state declarations. To increase carrying capacity and fuel efficiency, the Federal-Aid Highway Act Amendments of 1974 increased the GVW restriction to 80-kip as well as the single and tandem axle weights to 20- and 34-kip respectively. "As in the 1956 Act, these limits were permissive and States could adopt lower limits if they chose" (USDOT TS&W Study Vol-I 2000). The maximum weight two or more axle groupings for any vehicle could possess was determined by a bridge formula that utilizes a vehicles' weight-to-length ratio. This formula is currently in use and was created to provide safety and sustain the service life of bridges. The basic concept of the formula is to prevent overstressing HS-20 bridges by more than 5% and HS-15 bridges by more than 30% (USDOT TS&W Study—Working Paper 4 1995).

$$W = 500\left[\frac{LN}{N-1} + 12N + 36\right]$$

Federal Bridge Formula (a.k.a. "Formula B")

- W = the overall gross weight on any group of two or more consecutive axles to the nearest 500 pounds.
- L = the distance in feet between the outer axles of any group of two or more consecutive axles.
- N = the number of axles in the group under consideration.

Due to the lack of several states adopting the 80-kip weight limit, hence hindering carriers of states that adopted the 1974 limit, Congress enacted the Surface Transportation Assistance Act (STAA) of 1982 which mandated all states to practice and uphold the federal limits set in 1974 on Interstate Highways and other parts of the National Network (NN). STAA trucks are primarily classified as semitrailers with a minimum length of 48-ft and 28-ft (minimum) twin-trailers (USDOT TS&W Study Vol-I 2000).

The Intermodal Surface Transportation Efficiency Act of 1991 (ISTEA) along with the Transportation Efficiency Act of the 21st Century (TEA-21) put a freeze on state allowances of longer combination vehicles (LCVs). This limitation restricted the use of

LCVs in states that had not adopted the use of LCVs and prevented those currently in use from expanding LCV routes as well as LCV weights and dimensions. In contrast, state exemptions and grandfather rights regarding federal GVW limits can still be issued depending on certain criteria such as transporting goods that promote a state's economy (USDOT TS&W Study Vol-I 2000).

1.5 Previous Bridge Study

1.5.1 Impact of 44,000-kg (97,000-lb) Six-Axle Semitrailer Trucks on Bridges on Rural and Urban U.S. Interstate System

This study investigates the cost impacts that a proposed 97-kip six-axle truck would have on interstate bridges in the U.S. Over 37,500 simple and continuous span bridges were analyzed that were adequate for handling a typical five-axle semitrailer with a GVW of 80-kip (CS5). The effects are demonstrated by pairing the currently efficient bridges that become structurally deficient per the 97-kip configuration with the replacement costs and user costs. The replacement costs contain any cost accrued in raising bridge capacity to greater standards while user costs entail traffic congestion due to work zones. All bridge data were taken from the National Bridge Inventory (NBI). Using the previously developed technique from a FHWA project, the computerized "moment model" was used for analysis. By comparing the maximum positive/negative moments due to the live-load with those produced from the inventory ratings given in the NBI, the functionality of the bridges became apparent. Bridges were declared deficient only if the live load moment surpassed the inventory moment by 5%. The deck area of all deficient bridges were then quantified by state and multiplied by an average cost per deck surface area, depending on state location, determining partial strengthening costs.

Since these costs per deck area varied widely from state to state and other factors suggesting that strengthening a bridge will ultimately cost more than replacing the bridge, this study negated strengthening costs to replacement costs. User costs were quantified by time lost in work-zone congestion as well as additional fueling costs acquired in the traffic. By using the moment model and the work-zone analysis model, the following results and conclusions were determined.

- 38% of the bridges were declared deficient
- Deficient bridges were 56% rural and 44% urban
- Total replacement cost – $13.85 billion
- Rural replacement cost – $4.36 billion (31%)
- Urban replacement cost – $9.49 billion (69%)
- Total user cost – $56.07 billion
- Rural user cost – $6.55 billion (12%)
- Urban user cost – $49.51 billion (88%)

The 97-kip commercial vehicle will not be acceptable on almost 40% of the bridges on the U.S. Interstate Highway System that are currently equipped with the load carrying capacity that allows passage of the CS5 (80-kip) truck. On top of that, the replacement costs will increase above the value shown as bridges that are currently structurally deficient for the legal GVW of 80-kip must be replaced as well. A portion of this replacement cost should be added to the replacement cost for the six-axle truck. As replacement and user costs were the only variable cost associated with the impact heavier trucks have on bridges, additional impacts will be felt. The effect that vehicle emissions have on the environment during traffic congestion is an example. Due to the lack of specific data obtained from the NBI, detailed and complex models were not suitable for this study (Weissmann and Harrison 1998).

Chapter 2 Longitudinal Analysis & Results

2.1 Analysis Overview

To help catalogue the flexural and shear effects presented within this report, the results are organized in accordance to the structures' directional analysis types: longitudinal and transverse. Throughout the remainder of this section the term "longitudinal" shall be representative of the traffic flow direction while "transverse" corresponds to that along the cross-section. Analysis in the longitudinal direction is then separated according to bridge span support type: simply supported bridges and continuous span bridges. A single rigid pin, preventing vertical and axial translation, and a rigid roller, only preventing vertical translation, make up the support conditions for all simple spans. Each bridge was modeled with a pin at one end and a roller support at the other. All continuous span bridges consisted of two equal spans with pin-roller-roller supports at their respective ends. The effects included herein are the maximum internal bending moments and shear forces throughout each bridge due to the applied loadings. To ensure consistency, each bridge was modeled in two dimensions using a line-girder analysis. When considering the longitudinal force effects, the shears and moments are direct results from single vehicular live loads and do not contain any modification or design factors unless otherwise noted. However, additional factors are applied to the transverse force effects since ALDOT's Standard Bridge Slab design specifications will be checked against the proposed vehicles using LRFD design criteria. These modifications and the transverse results are later detailed in Chapter 3.

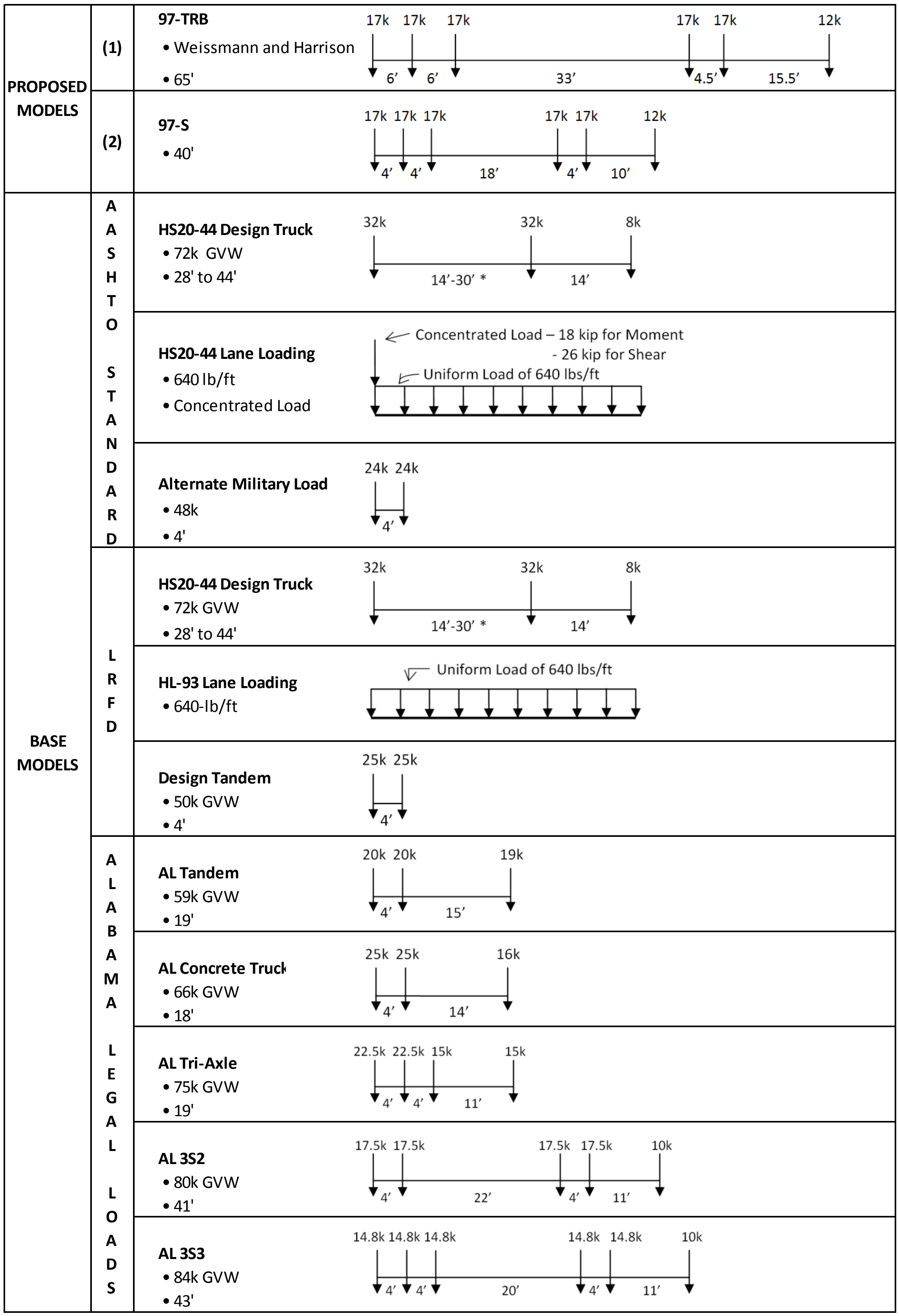
2.2 Vehicular Live Load Models

The force effects generated from two proposed vehicle configurations each with a GVW of 97-kip will be compared to the effects resulting from three base models. The number of axles of both 97-kip trucks is set at six but the variability between the two comes from individual axle spacing and the overall steering-to-rear-axle distance. While the 65-ft truck has been previously suggested by Weissmann and Harrison (1998), no literature was available regarding the shorter 40-ft configuration. Throughout the remainder of this report the 65-ft vehicle shall be referenced as “97-TRB” and the 40-ft short truck as “97-S”. All vehicular load models discussed throughout this section can be seen in Figure 2.1. The three base models used for comparison are not individual vehicles, but rather envelopes producing the maximum shear and moment effects from the design loadings detailed in AASHTO Standard Specifications for Highway Bridges design, AASHTO LRFD Bridge Design Specifications, and the envelope of five potentially critical Alabama Legal Loads.

The Standard Specifications utilize three highway live load scenarios for determining the critical design force effects. In general they are the notional HS20-44 design loading and an Alternate Military Loading (AML). The HS20-44 design loading is comprised of two separate load cases: the standard HS20-44 design truck and the HS20-44 design lane load. The HS20-44 design lane load includes a 640-lb/ft uniform load with a single concentrated load of 26-kip or 18-kip applied to the location causing the maximum force effect for shear or moment, respectively. The maximum force effect produced from one of the three loadings is taken as the design load (Article 3.7 AASHTO Standard Specifications 1996).

The LRFD Specifications use a vehicular live loading denoted HL-93, made up of three loadings: design truck, design tandem, and design lane load. Even though many similarities exist between both specifications, a few vital alterations are made. The design truck remains as the HS20-44 design truck. The AML is replaced with the design tandem axle loading which has a 2-kip increase in gross weight to that of the AML. The design lane load remains as 640-lb/ft but the additional concentrated loads are removed. The biggest difference from the design loadings of the Standard Specifications to LRFD is that the maximum force effect is the largest cumulative result of the design truck + design lane or design tandem + design lane loadings (Article 3.6.1.2 AASHTO LRFD Specifications 2010). It is shown in later figures that the force effects from LRFD loadings are comparatively greater than those of the standard specification.

The final base model includes selective legal loads that are specific to the State of Alabama. The five Alabama legal loads represented in this study are: Alabama Tandem-Axle, Alabama Concrete truck, Alabama Tri-Axle, Alabama 3S2, and the Alabama 3S3. As seen in Figure 2.1, the minimum steering-to-rear-axle spacing of the five vehicles is 18-ft while the maximum spacing is 43-ft. The minimum and maximum GVW of the Alabama legal loads range from 59-kip to 84-kip respectively. These vehicle configurations represent transport trucks that do not require permits to operate on Alabama highways.



* Axle spacing to vary producing maximum effect

Figure 2.1 – Vehicular Live Loads by Model Type

2.3 Simple Span Bridges

2.3.1 Methodology

Due to the determinant nature of simply supported bridges, line-girder analysis for determining the maximum force effects caused by vehicular live loads was achieved with the use of quantitative influence lines and spreadsheets generated in Microsoft Excel. Simple spans of 20-ft up to 300-ft are investigated with the 97-TRB and 97-S truck configurations. The resulting envelopes of the maximum force effects are then compared to those from the design live loads of the AASHTO Standard Specifications and the AASHTO LRFD Specifications. Comparisons of the proposed models to the effects of the five Alabama Legal Loads are made as well.

For simple spans, the maximum shear effect due to vehicular loads will always occur at a minuscule distance along the bridge span from one of the two supported ends and will have the greatest axle load bearing down on the support in question. The direction of vehicular travel is irrelevant as the truck is always positioned for its critical loading, creating maximum shear. By visualizing the influence line of arbitrary span length, one can easily determine the influence area ordinate under each adjacent axle by the use of similar triangles. This task is relatively simple for determinant structures, as influence lines maintain constant linear slopes.

In the case of determining the maximum moment for simple spans, it has been shown that the critical loading of a vehicle is at a position where the vehicles' resultant force and the adjacent concentrated axle load mirror the spans centerline. The axle closest to the resulting force usually dictates the maximum moment, but both adjacent axels should be checked (Hibbeler 2006). The location of the maximum moment will always be located directly under the governing axle load. Therefore, the maximum moment effects from all configurations are simple functions of vehicle geometry and span length. Referring to engineering terminology, this maximum moment is classified as positive since the extreme upper and lower fibers of the span's cross-section will be in compression and tension respectively. In LRFD design, the location of the maximum moment due to the vehicle and the uniform lane load will differ so both locations must be checked for each load. The cumulative design moment is then recorded for the single location producing the maximum effect.

2.3.2 Simple Span Force Effects

2.3.2.1 Shear at Supports

The longitudinal shear effect of each 97-kip vehicle is calculated as stated above and compared to the base design shears produced per the AASHTO Standard Specifications and the AASHTO LRFD Specifications as well as the five Alabama Legal Loads producing the maximum shear effect. The shear values resulting from these live loads are seen in Table 2-1. It is again noted that these are the maximum shear values located at either support of the simple span. In order to maintain space restraints, only the shear results for simple spans up to 150-ft are shown in the table. Results up to 300-ft are shown in the Appendix.

These results can be quite vague when only analyzing the shear values in their solitude. The shear ratios in Figure 2.2 – Figure 2.4 help clarify the functionality of these quantities. In each figure, the proposed-to-base shear ratio is less than 1.0 if the maximum resulting shear from the 97-kip vehicle falls below that of the base model and above 1.0 if not. For comparison purposes, ratios of both proposed live-load models are shown on the same graph.

Table 2-1: Simple Span–Maximum Shear due to Vehicular Loads

Span ft	97-S Kip	97-TRB kip	Standard kip	LRFD kip	AL Legal kip
20	40.80	35.70	43.20	51.40	50.25
25	42.84	38.76	46.08	54.08	55.20
30	46.47	40.80	49.60	59.20	58.50
35	51.97	42.26	52.80	64.00	60.86
40	56.10	43.35	55.20	68.00	62.63
45	60.64	44.20	57.07	71.47	64.00
50	64.28	46.75	58.56	74.56	65.10
55	67.25	50.23	59.78	77.38	66.00
60	69.73	53.13	60.80	80.00	66.75
65	71.83	55.58	61.66	82.46	67.38
70	73.63	58.54	62.40	84.80	67.93
75	75.19	61.10	63.04	87.04	68.40
80	76.55	63.34	63.60	89.20	68.81
85	77.75	65.32	64.09	91.29	69.18
90	78.82	67.08	64.53	93.33	69.50
95	79.78	68.66	64.93	95.33	69.79
100	80.64	70.08	65.28	97.28	70.05
105	81.42	71.36	65.60	99.20	70.29
110	82.13	72.52	65.89	101.1	70.50
115	82.77	73.59	66.16	103.0	70.99
120	83.37	74.56	66.40	104.8	71.54
125	83.91	75.46	66.62	106.6	72.04
130	84.42	76.29	67.60	108.4	72.50
135	84.88	77.06	69.20	110.2	72.92
140	85.31	77.77	70.80	112.0	73.32
145	85.72	78.43	72.40	113.8	73.69
150	86.09	79.05	74.00	115.5	74.03

Maximum shear from the 97-S exceeds the AASHTO Standard Specification in bridge spans ranging from approximately 40-ft to 200-ft (see Figure 2.2). This span range includes a large percentage of simply supported bridges in existence. A 20% increase in shear is seen for span lengths from 80-ft to 140-ft. The sharp increase in shear beginning at 40-ft spans is due to the 72-kip HS20-44 design truck controlling the design shear which is outweighed by the 97-S. At simple span lengths of 130-ft and greater, the design lane load becomes critical resulting in the decreasing slope of the shear curve.

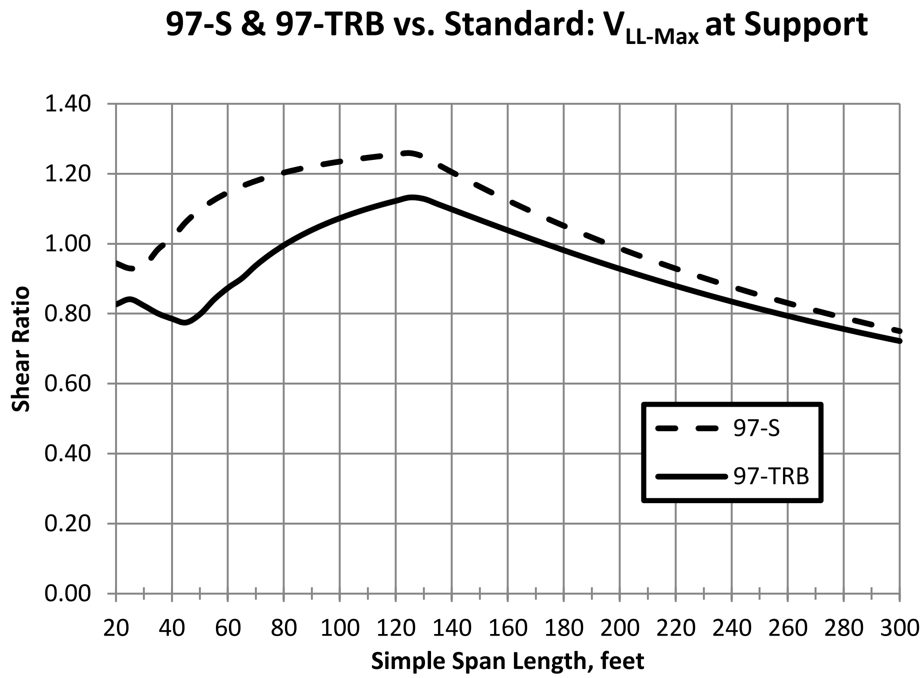


Figure 2.2 – Shear Ratio of 97-S & 97-TRB to AASHTO Standard Specifications

Maximum shear from the 65-ft 97-TRB exceeds the AASHTO Standard Specifications in bridge spans ranging from approximately 80-ft to 170-ft. The ratio curve strongly resembles that of the 97-S, however the maximum shear values shown from the 97-TRB are not as extreme. A 10% increase in shear results for spans lengths of 110-ft to 140-ft. The critical span length for both the 97-S and the 97-TRB is 125-ft, where the shear surges are 26% and 13% respectively.

The maximum shear from the both proposed trucks is completely enveloped by the LRFD design shear (see Figure 2.3). Notice that the maximum value of the 97-S is less than 90% of the design shear for 60’ spans and decreases as span length increases. This is due to the combination loading of the HL-93 design truck with the lane load.

The maximum shear effect of the 97-TRB is only 72% of the LRFD design. Comparing the ratios of the two proposed vehicles, it is seen that the maximum shear of the 97-S is greater than the 97-TRB for all simply supported spans. This is due to the shorter, more concentrated steering-to-real axle length of the 97-S.

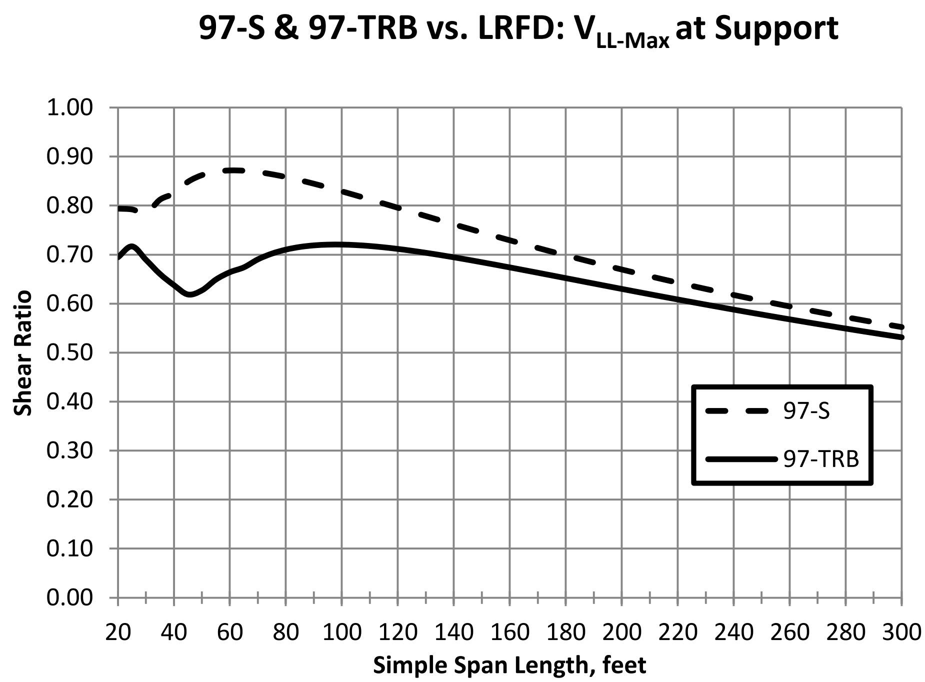


Figure 2.3 – Shear Ratio of 97-S & 97-TRB to LRFD Specifications

In comparison to the five Alabama Legal Loads (see Figure 2.4), the 97-S results in greater shear for all span lengths above 50-ft. For all spans 110-ft or longer a relatively constant shear ratio exists at 1.16. This can be explained by the 97-S's resemblance in axle grouping and overall length to the controlling legal load, AL 3S3. In addition, the GVW of the 97-S is 115% of the 84-kip AL 3S3.

Similar to the 97-S, as span length increases the 97-TRB causes more shear effects than the Alabama Legal Loads. However, the 97-S begins to exhibit greater shear at simple spans of 100-ft while the 97-S generates more shear beginning with spans of only 50-ft. The maximum shear increase in the 97-TRB is 11% while it is 17% for the

97-S loading. Comparing both proposed vehicles it is evident that shear effects not only increase with increasing truck weight but they are also dependent upon axle spacing. If axle spacing is minimized, shear effects will increase. The maximum shear at supports also approaches the gross vehicle weight as simple span length is increased.

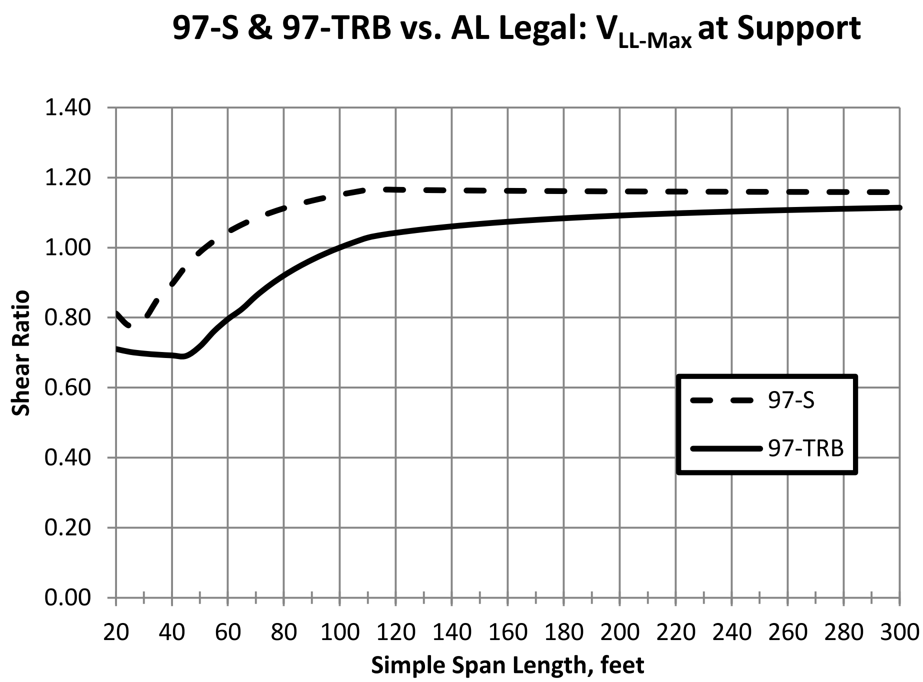


Figure 2.4 – Shear Ratio of 97-S & 97-TRB to AL Legal Loads

2.3.2.2 Maximum Positive Moment

The maximum moment resulting from the proposed and base models for simple spans of 20-ft to 150-ft is listed in Table 2-2. The moment values for spans up to 300-ft are listed in the Appendix. Recall that all moment values are positive due to the position of compression and tension zones of each member's hypothetical cross-section. The maximum moment ratio of the 97-S and 97-TRB to the base models is shown in Figures 2.5-2.7.

Compared to the unfactored design moments from the AASHTO Standard Specifications, the flexural effects of the 97-S loading create more moment in simply supported spans ranging from 50-ft to 205-ft in length (see Figure 2.5). In spans 105-ft to 155-ft long, a moment growth of 20% or more is shown. Regarding the Standard Specifications, the AML loading controls the design moment for simple spans up to 35-ft. The HS20-44 design truck then reins for spans extending to 140-ft where the design lane load governs for the remaining spans.

Table 2-2: Simple Span–Maximum Moment due to Vehicular Loads

Span ft	97-S kip-ft	97-TRB kip-ft	Standard kip-ft	LRFD kip-ft	AL Legal kip-ft
20	187.0	153.0	194.4	234.2	225.2
25	250.8	216.8	253.9	314.2	300.2
30	314.5	280.5	313.6	398.3	379.2
35	378.3	344.3	373.4	486.6	472.4
40	442.0	408.0	449.8	578.9	565.7
45	529.1	471.8	538.7	699.0	659.1
50	633.8	535.5	627.8	826.1	752.5
55	738.9	599.3	717.1	957.4	846.1
60	852.8	663.0	806.5	1093	939.6
65	971.6	726.8	896.0	1232	1033
70	1091	790.5	985.6	1376	1127
75	1210	872.7	1075	1523	1220
80	1330	977.0	1165	1675	1314
85	1450	1082	1255	1831	1408
90	1570	1186	1344	1991	1501
95	1690	1297	1434	2154	1595
100	1810	1415	1524	2322	1689
105	1930	1533	1614	2494	1782
110	2051	1652	1704	2670	1876
115	2171	1771	1793	2850	1970
120	2292	1890	1883	3034	2064
125	2412	2009	1973	3221	2157
130	2533	2129	2063	3413	2251
135	2654	2248	2153	3609	2345
140	2775	2368	2243	3809	2438
145	2895	2488	2335	4013	2532
150	3016	2607	2475	4221	2626

The 97-TRB creates more moment than the Standard Specifications in simple spans from 125-ft to 165-ft long. However, a 5% or more moment increase is only felt by spans from 140-ft to 150-ft long. The critical simple span length for both proposed vehicles is 145-ft, but the moment increase due to the 97-S is 24% over the positive moment effect from the Standard Specifications while the 97-TRB is 7%. Since the dominant variable of the proposed vehicles is the overall axle length, the maximum moment values begin to converge as the span length becomes infinite. The reasoning behind this can be explained by the ratio of moment arms of each configuration. With increasing span length the distance from the support to each vehicle's center of gravity becomes relatively comparable, treating the gross weight as a single point load.

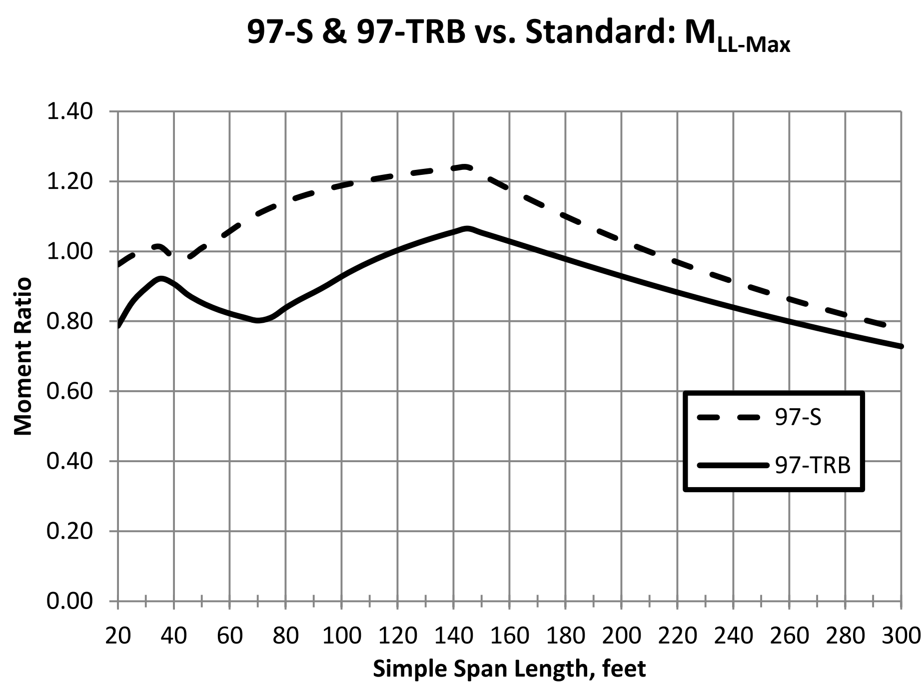


Figure 2.5 – Moment Ratio of 97-S & 97-TRB to AASHTO Standard Specifications

LRFD design moments completely envelop those of the 97-S and 97-TRB (see Figure 2.6). The maximum moment effect of the proposed vehicles is only 80% and 71%

of the value from the base LRFD model, respectively. Compared to the 97-S, the 97-TRB truck causes less moment effects on all simple spans.

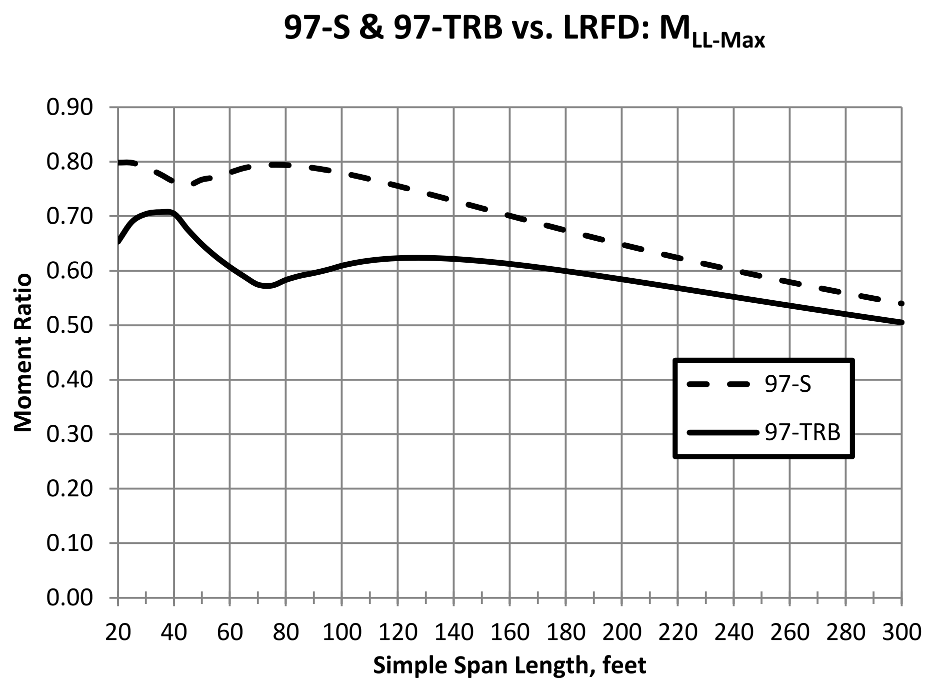


Figure 2.6 – Moment Ratio of 97-S & 97-TRB to LRFD Specifications

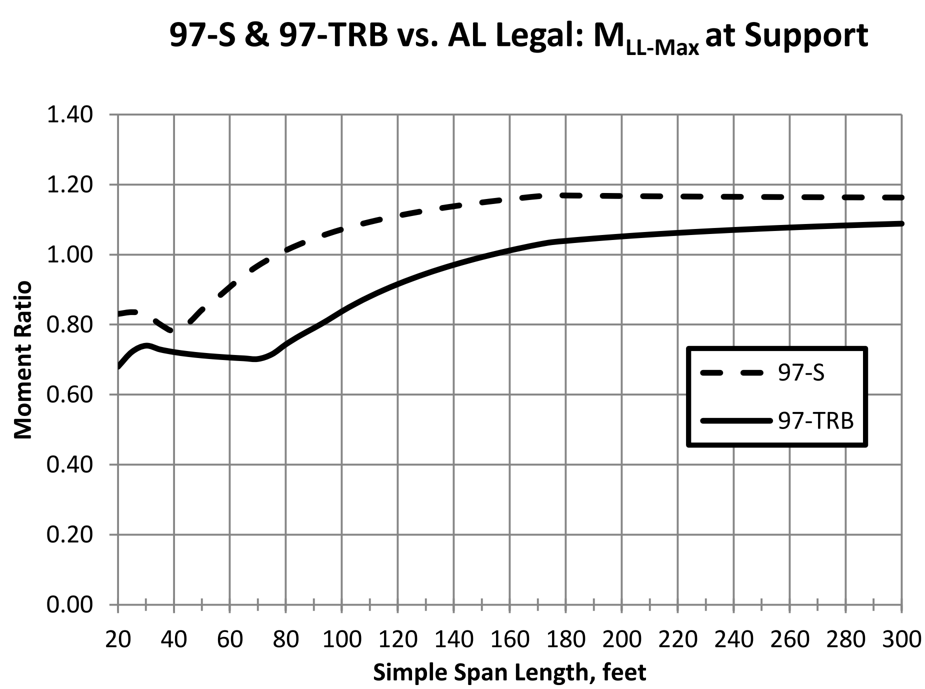


Figure 2.7 – Moment Ratio of 97-S & 97-TRB to AL Legal Loads

The 97-S exceeds the maximum bending moments caused by the Alabama Legal Loads on all simple spans of 80-ft and greater (see Figure 2.7). At spans around 180-ft, the moment increase becomes stable at 17% due to the AL 3S3 similarities previously discussed. At span lengths of 160-ft the 97-TRB will develop positive bending moments greater than the AL Legal Loads, specifically the Alabama Tri-Axle.

2.4 LRFD vs. AASHTO Standard Specifications Design Effects

The design live loads of the AASHTO Standard Specifications and the AASHTO LRFD Specifications contain many similarities as seen in Figure 2-1, but the unfactored force effects from each model vary considerably with the LRFD loadings giving conservative/higher results for design shear and moment. Figure 2.8 shows the increase in design values for simple spans with the numerical values listed in Tables 2-3 and 2-4.

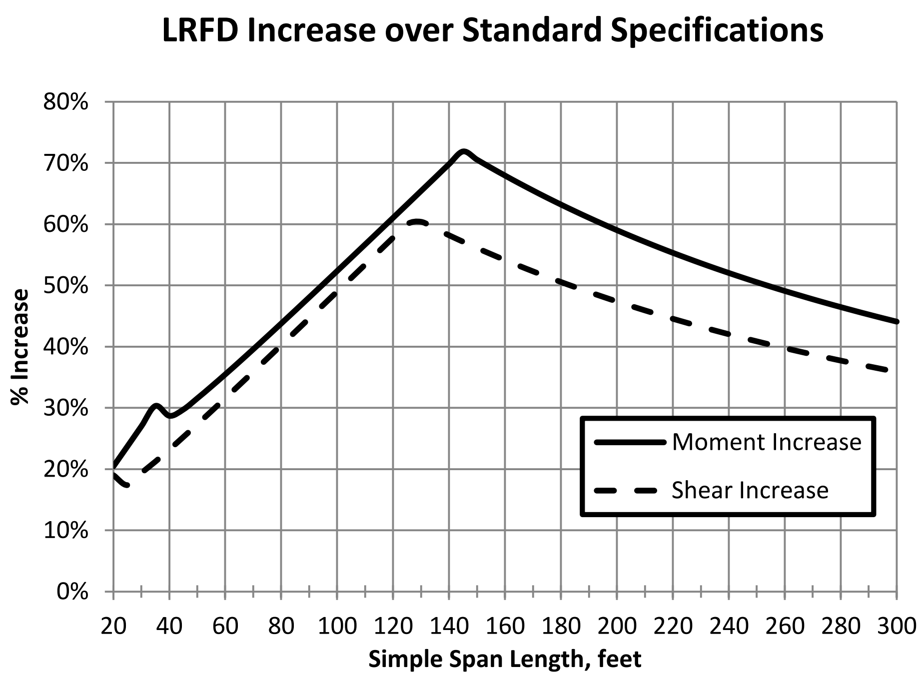


Figure 2.8 – Force Effect Increase from Design Loads of LRFD to Standard Specifications

The design loads from the AASHTO LRFD Specifications provide over 50% increases to the maximum shears resulting from the Standard Specifications ranging from 80-ft to 180-ft span lengths. 17% and 60% are the extreme parameters. A 50% increase in design moment occurs for spans from 100-ft to 250-ft long. The range of increase for the sample spans is 20% - 72%.

Table 2-3: Simple Span–LRFD Shear Increase to AASHTO Standard Specifications

Span ft	Standard kip	LRFD kip	**LRFD Increase**	Span ft	Standard kip	LRFD kip	**LRFD Increase**
20	43.20	51.40	**19%**	165	78.80	120.7	**53%**
25	46.08	54.08	**17%**	170	80.40	122.4	**52%**
30	49.60	59.20	**19%**	175	82.00	124.2	**51%**
35	52.80	64.00	**21%**	180	83.60	125.9	**51%**
40	55.20	68.00	**23%**	185	85.20	127.6	**50%**
45	57.07	71.47	**25%**	190	86.80	129.3	**49%**
50	58.56	74.56	**27%**	195	88.40	131.0	**48%**
55	59.78	77.38	**29%**	200	90.00	132.6	**47%**
60	60.80	80.00	**32%**	205	91.60	134.3	**47%**
65	61.66	82.46	**34%**	210	93.20	136.0	**46%**
70	62.40	84.80	**36%**	215	94.80	137.7	**45%**
75	63.04	87.04	**38%**	220	96.40	139.3	**45%**
80	63.60	89.20	**40%**	225	98.00	141.0	**44%**
85	64.09	91.29	**42%**	230	99.60	142.7	**43%**
90	64.53	93.33	**45%**	235	101.20	144.3	**43%**
95	64.93	95.33	**47%**	240	102.80	146.0	**42%**
100	65.28	97.28	**49%**	245	104.40	147.7	**41%**
105	65.60	99.20	**51%**	250	106.00	149.3	**41%**
110	65.89	101.1	**53%**	255	107.60	151.0	**40%**
115	66.16	103.0	**56%**	260	109.20	152.6	**40%**
120	66.40	104.8	**58%**	265	110.80	154.3	**39%**
125	66.62	106.6	**60%**	270	112.40	155.9	**39%**
130	67.60	108.4	**60%**	275	114.00	157.6	**38%**
135	69.20	110.2	**59%**	280	115.60	159.2	**38%**
140	70.80	112.0	**58%**	285	117.20	160.8	**37%**
145	72.40	113.8	**57%**	290	118.80	162.5	**37%**
150	74.00	115.5	**56%**	295	120.40	164.1	**36%**
155	75.60	117.3	**55%**	300	122.00	165.8	**36%**
160	77.20	119.0	**54%**				

Table 2-4: Simple Span–LRFD Moment Increase to AASHTO Standard Specifications

Span ft	Standard kip-ft	LRFD kip-ft	**LRFD Increase**	Span ft	Standard kip-ft	LRFD kip-ft	**LRFD Increase**
20	194.4	234.2	**20%**	165	2921	4869	**67%**
25	253.9	314.2	**24%**	170	3077	5093	**66%**
30	313.6	398.3	**27%**	175	3238	5320	**64%**
35	373.4	486.6	**30%**	180	3402	5552	**63%**
40	449.8	578.9	**29%**	185	3571	5788	**62%**
45	538.7	699.0	**30%**	190	3743	6028	**61%**
50	627.8	826.1	**32%**	195	3920	6272	**60%**
55	717.1	957.4	**34%**	200	4100	6520	**59%**
60	806.5	1093	**35%**	205	4285	6772	**58%**
65	896.0	1232	**38%**	210	4473	7028	**57%**
70	985.6	1376	**40%**	215	4666	7288	**56%**
75	1075	1523	**42%**	220	4862	7552	**55%**
80	1165	1675	**44%**	225	5063	7820	**54%**
85	1255	1831	**46%**	230	5267	8092	**54%**
90	1344	1991	**48%**	235	5476	8368	**53%**
95	1434	2154	**50%**	240	5688	8648	**52%**
100	1524	2322	**52%**	245	5905	8932	**51%**
105	1614	2494	**55%**	250	6125	9220	**51%**
110	1704	2670	**57%**	255	6350	9512	**50%**
115	1793	2850	**59%**	260	6578	9808	**49%**
120	1883	3034	**61%**	265	6811	10108	**48%**
125	1973	3221	**63%**	270	7047	10412	**48%**
130	2063	3413	**65%**	275	7288	10720	**47%**
135	2153	3609	**68%**	280	7532	11032	**46%**
140	2243	3809	**70%**	285	7781	11348	**46%**
145	2335	4013	**72%**	290	8033	11668	**45%**
150	2475	4221	**71%**	295	8290	11992	**45%**
155	2620	4433	**69%**	300	8550	12320	**44%**
160	2768	4649	**68%**				

2.5 Continuous Span Bridges

2.5.1 Methodology

Evaluating the internal force effects in continuous bridges can be a tedious process using hand calculations since the bridges are statically indeterminate to the first degree as verified by their pin-roller-roller supports previously discussed. To maximize efficiency, all continuous span bridges are analyzed using CSI's SAP2000 V15.

SAP2000 is a powerful software tool used by many structural engineers worldwide for design and analysis purposes. The maximum force effects from the proposed and base vehicles are presented in the same manner as they are for the simply supported bridges. The two-span continuous bridges have span lengths ranging from 20-ft to 150-ft with a span ratio of 1:1. Using the moving load feature in SAP2000, each bridge model is effectively loaded and analyzed under linear static load conditions. The maximum discretization length for the vehicle loadings is set at one-foot or one-one-hundredth of the span length with the smallest value controlling the discretization length. All supports are assumed rigid.

For continuous spans, the critical locations of maximum shearing force occur at the three bridge supports. For two-span continuous bridges with a span ratio of 1:1, the center support experiences the greatest shearing effect.

Two maximum moment effects are vital in continuous spans. These can be clarified as the maximum positive moment and the maximum negative moment. The positive moment refers to the bending phenomena that cause the top fibers of the span's cross-section to experience compression. The location of maximum positive moments varies depending on the span length of the bridge and the vehicle's load configuration but usually occurs at a distance within 35%-45% of the span length. The negative moment creates tensile forces in the top fibers of the span's cross-section. The location of the maximum negative moment for two-span continuous bridges is always about the center support of the bridge. Referring to the HS20-44 design lane load in the AASHTO Standard Specifications, the maximum negative design moment for continuous spans is determined by modifying the lane load to include an additional 18-kip concentrated load (one in each span) to produce the maximum effect per Article 3.11.3. In the LRFD Specifications, the maximum negative moment in continuous spans is described in Article 3.6.1.3. In summary, it denotes that negative moment is calculated as 90% of the effect of two design trucks spaced 50-ft apart (i.e. rear axle of truck one and front axle of truck two) combined with 90% of the lane load.

2.5.2 Continuous Span Force Effects

2.5.2.1 Shear at End Supports

The maximum shear in the end supports from the proposed and base models is given in Table 2-5. The resulting shear ratio curves comparing the 97-S and the 97-TRB to the base models are presented in Figures 2.9 –2.11.

The 97-S causes more shear than the AASHTO Standard Specifications at the end supports for continuous spans greater than 40-ft x 40-ft. 90-ft x 90-ft spans and longer experience shear increases of 20% or more (see Figure 2.9). The maximum increase of 25% will be felt for 1:1 span ratios of 145-ft. However, the 97-TRB only exceeds shear values of the Standard Specifications for spans of 95-ft or greater and the maximum increase (12%) is less than half the increase of the 97-S. Span lengths ranging from 130-ft to 150-ft will result in shear growth slightly above 10% when traveled by the 97-TRB. As both curves show, the maximum increase in shear slightly drops at 145-ft spans, suggesting that this is the critical span length for the shear initiated by both 97-kip loads.

Table 2-5: Continuous Span–Maximum Shear at End Supports

Span ft	97-S kip	97-TRB kip	Standard kip	LRFD kip	AL Legal kip
20x20	38.55	32.91	42.03	55.05	47.55
25x25	40.95	36.22	43.21	56.56	52.03
30x30	43.93	38.55	46.43	58.03	55.40
35x35	48.18	40.26	49.52	59.48	57.96
40x40	51.87	41.56	52.02	63.21	59.96
45x45	55.76	42.58	54.05	66.64	61.55
50x50	59.16	44.47	55.72	69.71	62.84
55x55	62.12	47.01	57.11	72.50	63.91
60x60	64.69	49.37	58.30	75.09	64.82
65x65	66.93	51.54	59.31	77.50	65.58
70x70	68.91	53.98	60.19	79.77	66.25
75x75	70.64	56.24	60.95	81.94	66.82
80x80	72.19	58.32	61.62	84.01	67.32
85x85	73.57	60.23	62.22	86.01	67.77
90x90	74.81	61.98	62.75	87.94	68.16
95x95	75.92	63.60	63.23	89.82	68.52
100x100	76.94	65.09	63.68	91.67	68.85
105x105	77.86	66.45	64.06	93.45	69.14
110x110	78.71	67.71	64.42	95.20	69.40
115x115	79.48	68.89	64.75	96.90	69.65
120x120	80.19	69.97	65.04	98.6	69.86
125x125	80.84	70.97	65.32	100.3	70.07
130x130	81.46	71.91	65.58	102.0	70.26
135x135	82.02	72.78	65.82	103.6	70.44
140x140	82.55	73.61	66.04	105.2	70.81
145x145	83.03	74.37	66.61	106.8	71.25
150x150	83.51	75.10	68.02	108.4	71.67

The maximum shear from the LRFD design loads envelopes both proposed 97-kip models (see Figure 2.10). At the bridge abutment/span end interface, the 97-S will produce shear values less than 90% of the LRFD shears while the shears from the 97-TRB are considerably lower at 71%. As noted in the shear analysis of the simple span bridges, the shorter overall axle spacing of the 97-S of 40-ft proves to be the critical parameter for the proposed vehicles. The maximum shear values from the LRFD design loads are increasing at a greater proportion than the 97-kip loadings as seen by the negative slopes in the longer spans in each plot. This is expected since the LRFD loads include both the design truck and the uniform lane load.

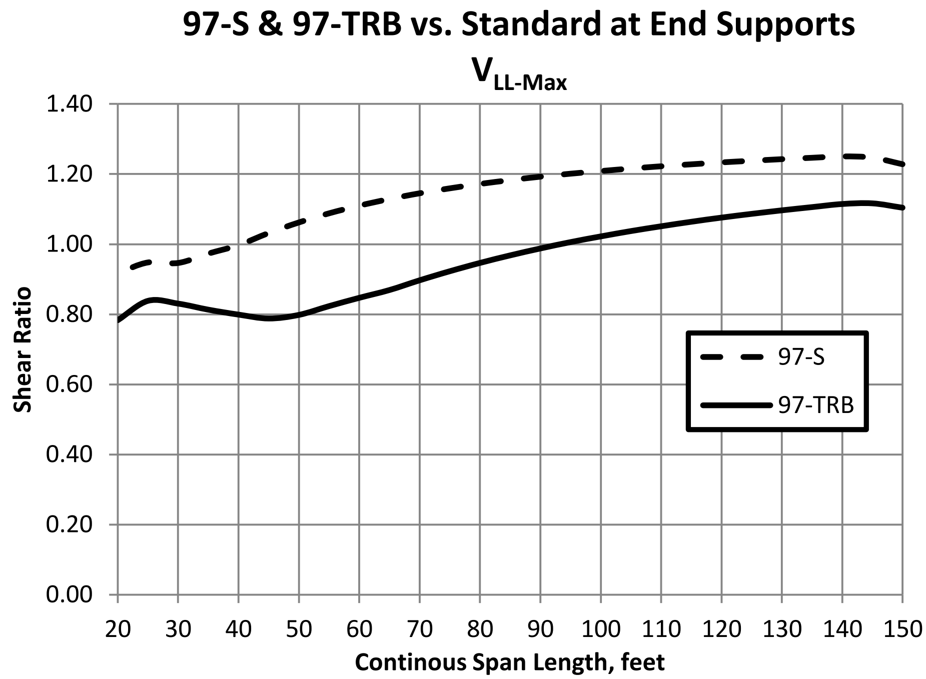


Figure 2.9 – Shear Ratio at End Supports of 97-S & 97-TRB to AASHTO Standard Specifications

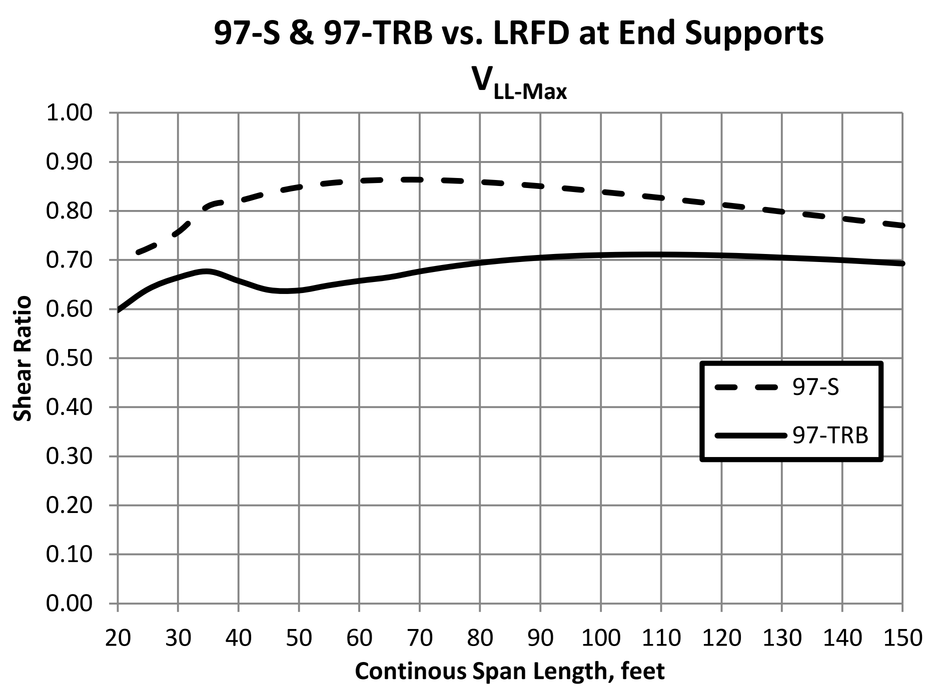


Figure 2.10 – Shear Ratio at End Supports of 97-S & 97-TRB to LRFD Specifications

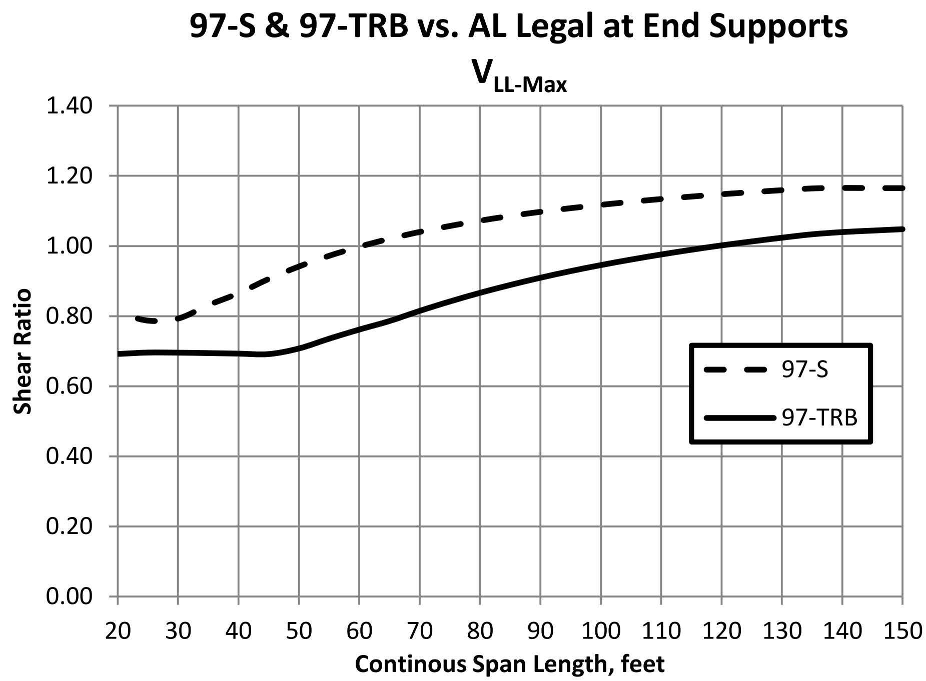


Figure 2.11 – Shear Ratio at End Supports of 97-S & 97-TRB to AL Legal Loads

The shear from the 97-S is greater than those of the AL Legal Loads for continuous spans greater than 60-ft. The maximum shear increase becomes constant at 17% for spans 140-ft and greater. This 17% increase can be explained by the difference in gross vehicle weight of the 97-S and the 84-kip AL 3S3. Due to the greater 65-ft length of the 97-TRB, the shear effects are lower than the 97-S because the axle loads are not as compressed and are distributed to all supports at greater proportions. The 97-TRB begins to exhibit greater shear effects than the AL Legal Loads at spans above 120-ft. The constant shear increase approaches 5% as bridge spans lengthen.

2.5.2.2 Shear at Center Support

The maximum shear at the center support from the proposed and base models is given in Table 2-6. The resulting shear ratios comparing the 97-S and the 97-TRB to the base models are presented in Figures 2.12 –2.14.

Table 2-6: Continuous Span–Maximum Shear at Center Support

Span ft	97-S kip	97-TRB kip	Standard kip	LRFD kip	AL Legal kip
20x20	44.04	38.61	44.90	57.76	53.45
25x25	47.01	41.71	49.60	59.80	58.77
30x30	49.35	43.64	52.86	64.84	62.15
35x35	55.70	45.17	56.23	70.22	64.43
40x40	60.33	46.84	58.73	74.71	66.08
45x45	65.47	48.28	60.59	78.58	67.30
50x50	69.49	49.41	62.03	82.01	68.24
55x55	72.68	53.45	63.17	85.14	68.99
60x60	75.26	56.78	64.08	88.06	69.59
65x65	77.39	59.54	64.84	90.81	70.09
70x70	79.17	62.94	65.46	93.44	70.51
75x75	80.68	65.84	66.00	95.97	70.86
80x80	81.96	68.33	66.45	98.42	71.17
85x85	83.08	70.49	66.85	100.8	71.43
90x90	84.05	72.39	67.19	103.2	71.94
95x95	84.89	74.05	67.49	105.5	72.73
100x100	85.65	75.52	67.76	107.7	73.44
105x105	86.31	76.83	68.00	110.0	74.06
110x110	86.91	78.01	69.98	112.2	74.62
115x115	87.45	79.06	71.98	114.4	75.12
120x120	87.93	80.01	73.97	116.5	75.58
125x125	88.37	80.88	75.97	118.7	75.99
130x130	88.77	81.66	77.97	120.8	76.36
135x135	89.14	82.38	79.97	123.0	76.71
140x140	89.48	83.04	81.97	125.1	77.02
145x145	89.79	83.64	83.97	127.1	77.31
150x150	90.07	84.20	85.96	129.3	77.58

The 97-S demonstrates more shear force than the design loads from the AASHTO Standard Specifications produce at the center support for bridge spans 40-ft to 150-ft long (see Figure 2.12). Continuous spans of 70-ft to 115-ft have a 20% increase or more in shear values compared to the Standard Specifications. The critical span length is 105-ft where the shear increase is 27%. The maximum shear values resulting from the 97-TRB are greater for continuous spans of 80-ft to 140-ft in length. At span lengths of

95-ft to 115-ft, the shear increase is at or above 10% of the Standard Specifications. The critical span length is 105-ft where the shear increase is 13%.

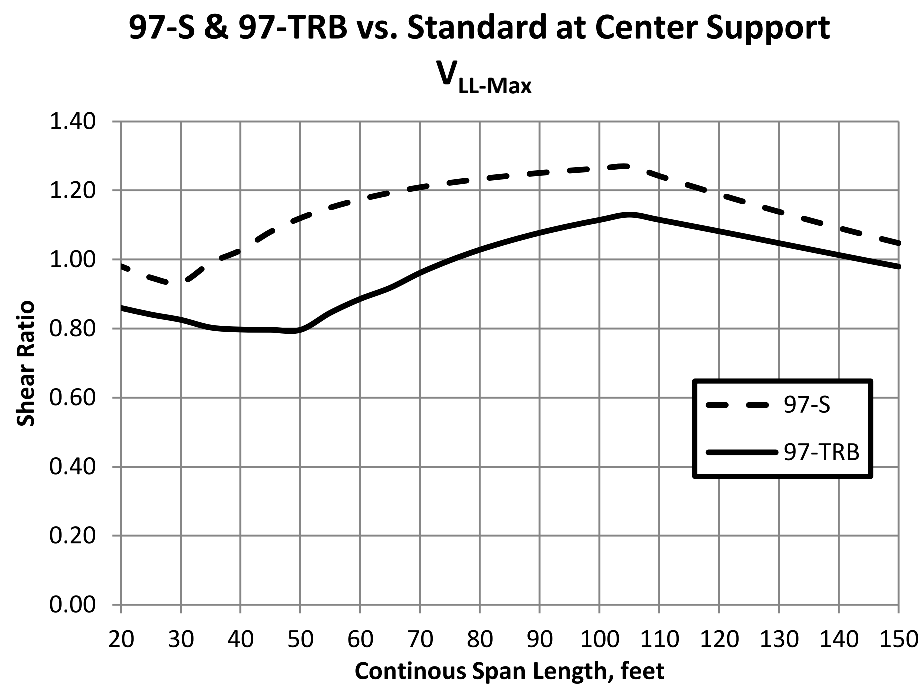


Figure 2.12 – Shear Ratio at Center Support of 97-S & 97-TRB to AASHTO Standard Specifications

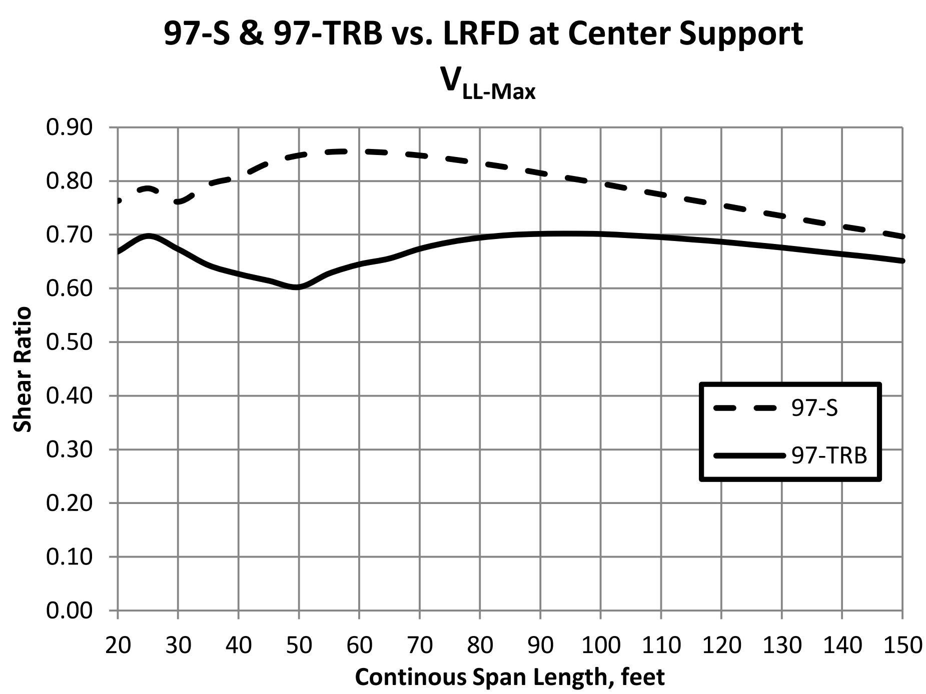


Figure 2.13 – Shear Ratio at Center Support of 97-S & 97-TRB to LRFD Specifications

The maximum shear from the LRFD design loads envelopes both proposed 97-kip models (see Figure 2.13). At locations about the center support, the 97-S will produce shear values less than 90% of the LRFD shears while the shears from the 97-TRB are considerably lower at a maximum of 70%. The shear ratios also decline as the bridge span increases; suggesting LRFD design loads produce conservative/higher design effects as for longer span bridges.

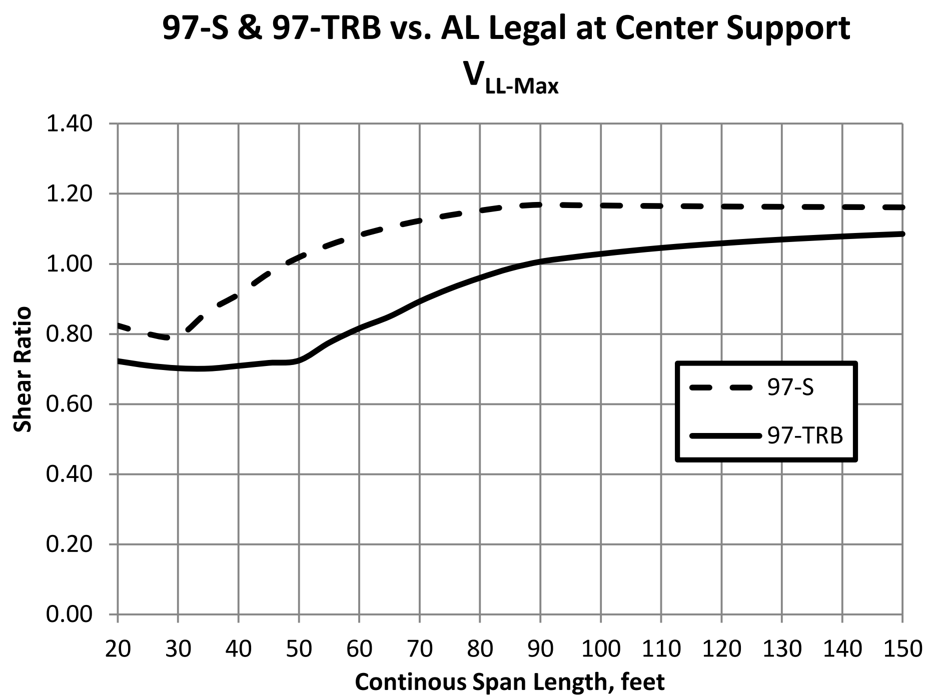


Figure 2.14 – Shear Ratio at Center Support of 97-S & 97-TRB to AL Legal Loads

The controlling shear from the 97-S is greater than those of the AL Legal Loads for continuous spans of 50-ft or greater. The maximum shear increase becomes constant at 16-17% for spans 85-ft and greater. The 97-TRB begins to exhibit shear forces greater than the AL Legal Loads at continuous spans of 90-ft and longer. The constant shear increase approaches 10% as bridge spans lengthen.

2.5.2.3 Maximum Positive Moment

The maximum positive moment effects from the proposed and base live load models are given in Table 2-7. The location of these moments varies between 35%-45% of the span length. The corresponding moment ratio curves are shown in Figure 2.15-2.17.

Table 2-7: Continuous Span–Maximum Positive Moment

Span ft	97-S kip-ft	97-TRB kip-ft	Standard kip-ft	LRFD kip-ft	AL Legal kip-ft
20x20	147.7	118.8	157.7	228.2	179.1
25x25	199.8	169.7	206.6	293.7	240.3
30x30	252.1	221.4	255.7	362.4	308.4
35x35	304.6	273.4	305.0	434.1	382.1
40x40	362.1	325.6	358.4	508.9	456.9
45x45	432.5	378.0	429.2	586.7	532.5
50x50	510.3	430.5	500.8	667.6	608.5
55x55	590.2	483.1	572.9	757.4	684.9
60x60	681.9	535.7	645.4	865.3	761.6
65x65	775.0	588.3	718.4	976.4	838.3
70x70	869.3	647.9	791.7	1091	915.2
75x75	964.6	721.1	865.1	1208	992.3
80x80	1061	797.8	938.7	1329	1070
85x85	1157	876.0	1012	1454	1147
90x90	1254	955.3	1086	1581	1224
95x95	1351	1042	1160	1712	1302
100x100	1449	1134	1234	1845	1379
105x105	1547	1226	1308	1982	1456
110x110	1645	1320	1382	2122	1534
115x115	1744	1414	1456	2265	1611
120x120	1842	1508	1530	2411	1689
125x125	1941	1603	1605	2560	1767
130x130	2040	1699	1679	2712	1844
135x135	2139	1795	1753	2868	1922
140x140	2238	1891	1828	3026	1999
145x145	2337	1988	1902	3188	2077
150x150	2437	2084	1976	3353	2155

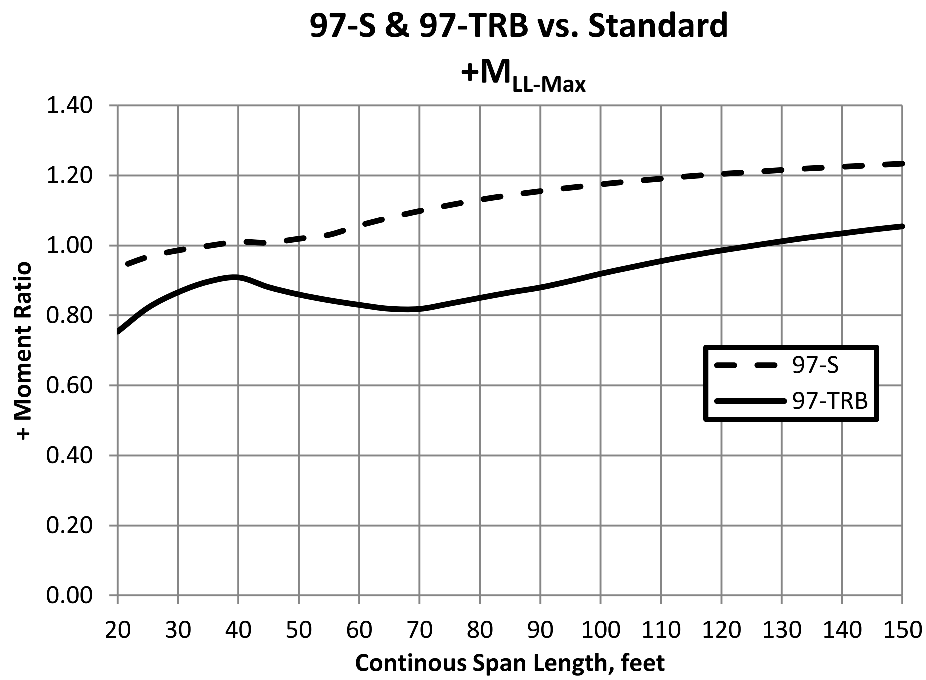


Figure 2.15 – Positive Moment Ratio of 97-S & 97-TRB to AASHTO Standard Specifications

The maximum bending moment of the 97-S exceeds the moment from the Standard Specifications for continuous spans of 40-ft and greater. The increase in moment is proportional to span lengths up to 150-ft. At span lengths of 115-ft and longer, the increase is greater than or equal to 20%. Effects of the 97-TRB are not as drastic, but bridges 130-ft and greater in span length will begin to experience moments greater than those from the Standard Specifications. The critical continuous span length for both proposed trucks is 150x150-ft where the 97-S and the 97-TRB experience positive moment increases of 23% and 5% respectively. However, the ratio curve from both 97-kip trucks appears to be increasing at 150x150-ft spans, so spans exceeding 150-ft in length should be checked to determine the actual critical span length.

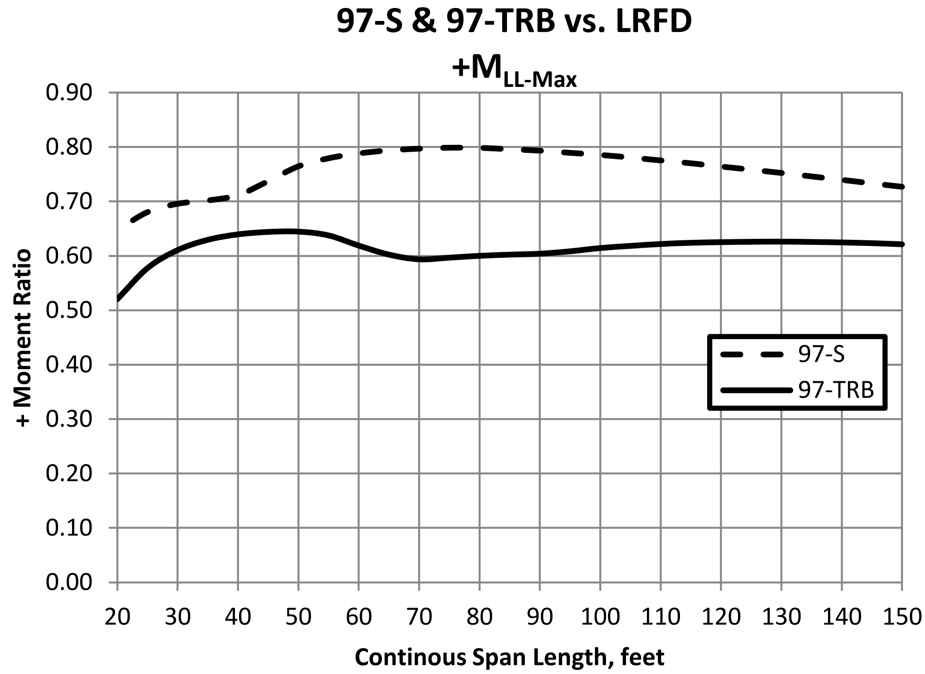


Figure 2.16 – Positive Moment Ratio of 97-S & 97-TRB to LRFD Specifications

Once again the LRFD design loads produce force effects that envelop all effects from the 97-S and the 97-TRB. The maximum positive moment of the 97-S is only 80% of the design value where the maximum value of the 97-TRB is only 63% of that established by LRFD loads.

At spans of 85-ft, the 97-S causes an increase in bending moment over that of the AL Legal Loads (see Figure 2.17). The maximum positive moment increase due to the 97-S is 13% at the longest continuous span analyzed of 150'x150'. In regards to the 97-TRB, the AL Legal Loads produce greater positive moments for every continuous span length included in this study. The critical vehicle configuration from the Alabama Legal Loads is the 75-kip Alabama tri-axle due to its short overall axle length of 19-ft combined with the 60-kip tri-axle. As noted previously, axle spacing has a significant impact on the magnitude of the force effect developed in bridge members.

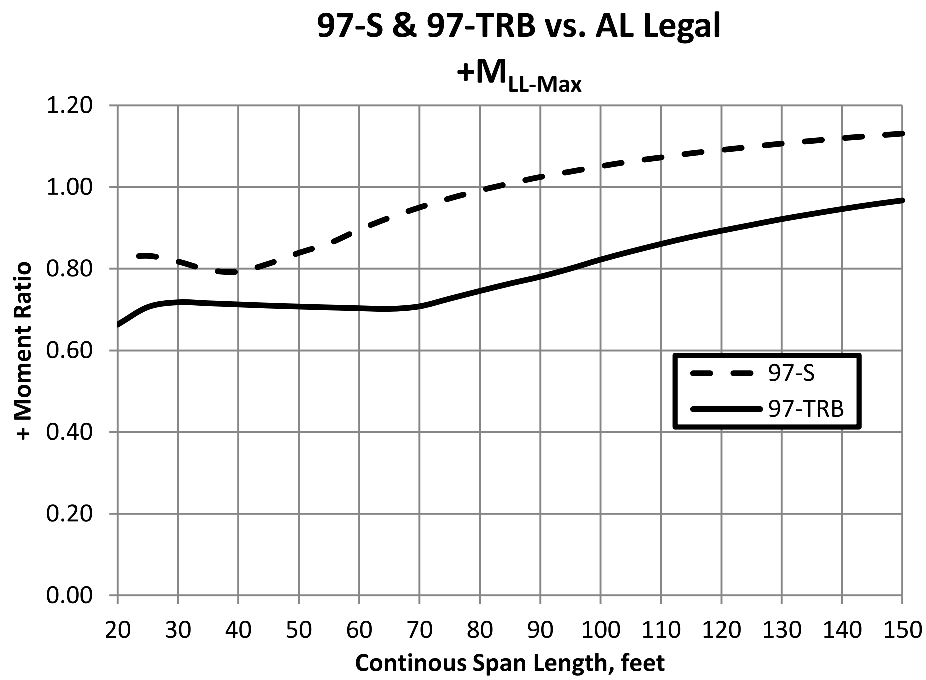


Figure 2.17 – Positive Moment Ratio of 97-S & 97-TRB to AL Legal Loads

2.5.2.4 Maximum Negative Moment

For continuous spans, the location of the critical negative moment occurs at interior supports. The results of the maximum negative moment of the five load models are given in Table 2-8. The corresponding moment ratios are shown in Figure 2.18-2.20.

The 97-S truck results in negative bending moments above those from the Standard Specification for continuous span lengths shorter than 80x80-ft (see Figure 2.18). The maximum negative moment increase is 31% at the critical span length of 30x30-ft. Continuous spans of 25-ft to 55-ft have a 20% increase or more in shear values compared to the Standard Specifications. The 97-TRB vehicle produces values above the design values for span ranges of 35-ft to 75-ft. The maximum negative moment increase is 41% at the critical span length of 55x55-ft. Continuous spans from 45-ft to 65-ft in length exhibit a 20% increase or more in shear values compared to the Standard

Specifications. This is also the first plot that demonstrates force effects from the 97-TRB above those of the 97-S. This takes place in continuous span lengths ranging from 45-ft to 80-ft. Both proposed vehicle models generate negative moments appreciably lower than the standard design as span length increases.

Table 2-8: Continuous Span–Maximum Negative Moment

Span ft	97-S kip-ft	97-TRB kip-ft	Standard kip-ft	LRFD kip-ft	AL Legal kip-ft
20x20	132.0	84.14	122.3	154.0	129.8
25x25	194.4	105.2	155.8	205.5	161.7
30x30	253.1	170.0	192.5	264.2	215.7
35x35	299.6	241.7	228.9	326.5	260.5
40x40	337.1	308.3	264.3	392.5	297.0
45x45	367.6	374.5	294.6	459.7	326.8
50x50	393.2	431.6	320.0	525.6	351.8
55x55	414.6	481.2	341.5	653.1	372.9
60x60	432.9	524.5	391.5	806.1	409.3
65x65	483.7	562.5	450.2	958.9	447.1
70x70	538.3	596.0	512.8	1106	484.6
75x75	591.9	625.7	579.5	1248	522.0
80x80	644.8	652.3	650.2	1385	559.2
85x85	697.0	676.2	724.9	1519	596.4
90x90	748.7	697.8	803.5	1651	636.5
95x95	799.9	717.4	886.2	1780	681.4
100x100	850.6	735.4	972.8	1909	725.8
105x105	901.1	767.8	1063	2036	769.9
110x110	951.1	823.1	1158	2163	813.7
115x115	1001	877.8	1257	2290	857.3
120x120	1050	931.8	1359	2417	900.6
125x125	1100	985.4	1466	2545	943.6
130x130	1149	1038	1577	2674	986.5
135x135	1198	1091	1691	2805	1029
140x140	1247	1143	1810	2936	1072
145x145	1295	1195	1933	3070	1114
150x150	1344	1247	2059	3205	1156

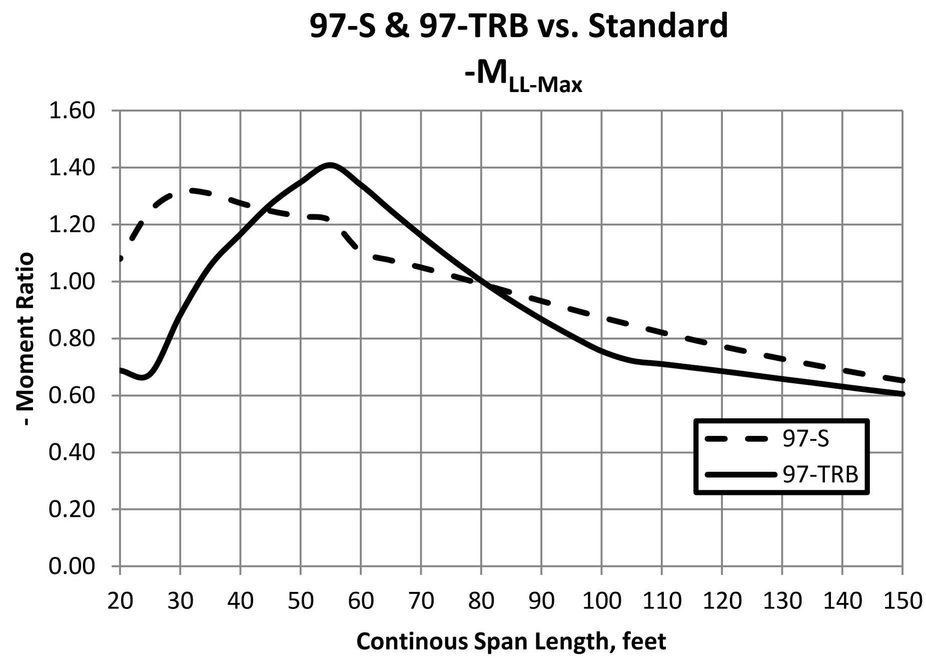


Figure 2.18 – Negative Moment Ratio of 97-S & 97-TRB to AASHTO Standard Specifications

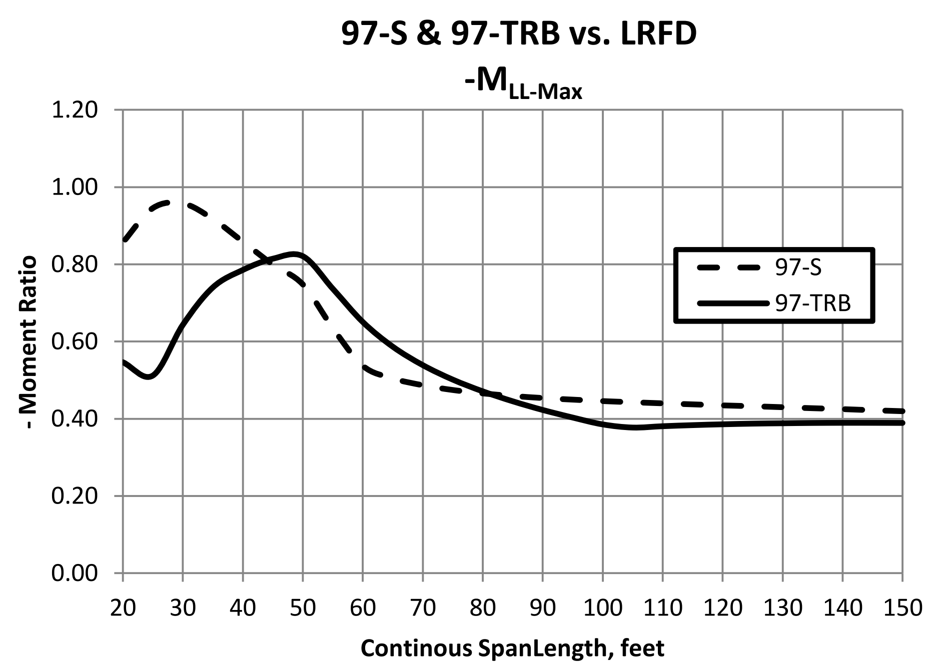


Figure 2.19 – Negative Moment Ratio of 97-S & 97-TRB to LRFD Specifications

Keeping consistent with previous results, the negative moment developed by the LRFD design loadings envelop both proposed 97-kip vehicles (see Figure 2.19). At the critical continuous span length of 30x30-ft, the moment caused by the 97-S is only 96% of the effect from maximum design loading of the AASHTO LRFD Specifications. For the 97-TRB, the critical moment is only 82% for a 50x50-ft bridge. As span length increases the proposed models establish negative moments that approach constant ratios near 40% of the LRFD design moments.

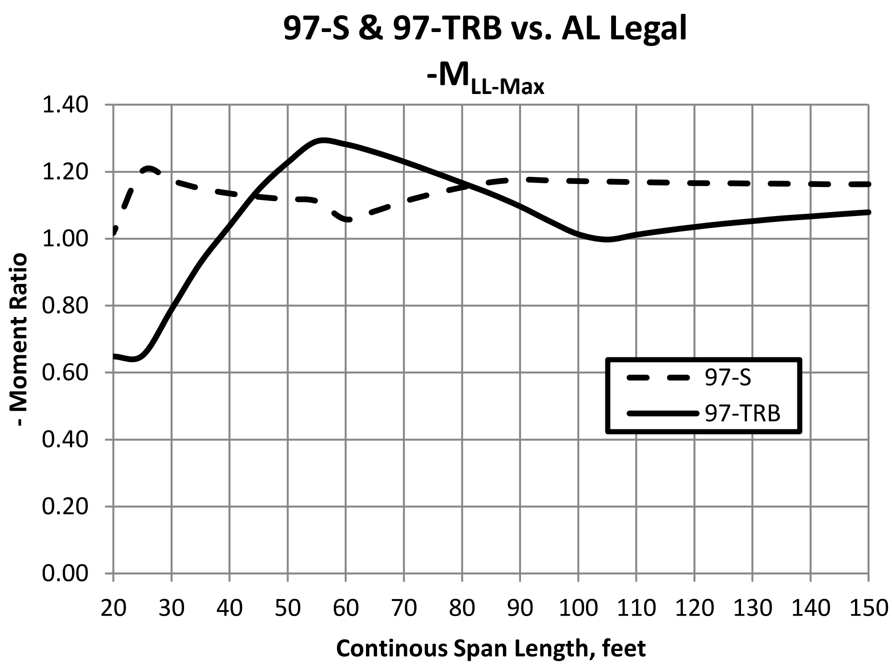


Figure 2.20 – Negative Moment Ratio of 97-S & 97-TRB to AL Legal Loads

The 97-S will cause critical negative moments above those of the AL Legal Loads for all continuous span bridges included in this study. At span lengths above 80-ft, the variability in the negative moment ratio diminishes as a constant increase of 17% forms. Moments developed from the 97-TRB are greater than those from the AL Legal Loads on continuous spans 40x40-ft and greater.

CHAPTER 3 TRANSVERSE DECK ANALYSIS & RESULTS

3.1 Analysis Overview

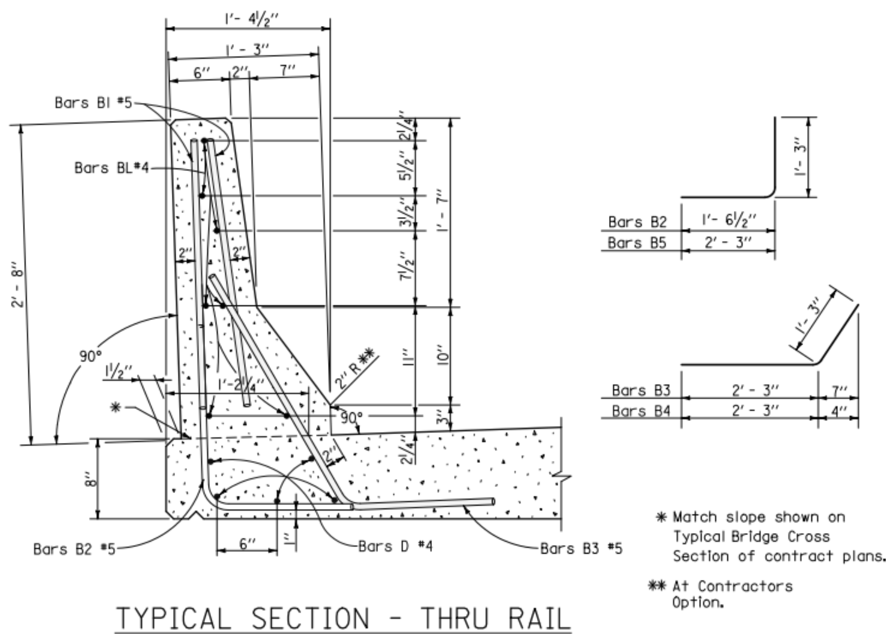
This section contains the transverse effects that reinforced concrete deck slabs in slab-girder bridges must withstand when traveled by the proposed 97-kip models (see Figure 2.1). Decks that are supported by longitudinal girders having aspect ratios of 1.5 or greater can be considered one-way slab systems. The aspect ratio is defined as the longitudinal span distance between supports divided by the transverse girder spacing (Barker and Puckett 2007). All bridge models in this study meet this minimum criterion so it is justified to treat each deck as a continuous beam

Moment effects are critical over shearing forces in deck design. This is due to the limited flexural stiffness in the concrete deck sections spanning between girders. For this reason, only positive and negative moment conditions are recorded. Using LRFD techniques, the ultimate factored design moment was checked against the nominal moment capacity or resistance supplied by the reinforced concrete decks outlined in ALDOT's standard slab details. All referenced articles within this section refer to the AASHTO LRFD Bridge Design Specifications.

3.2 ALDOT Bridge Design

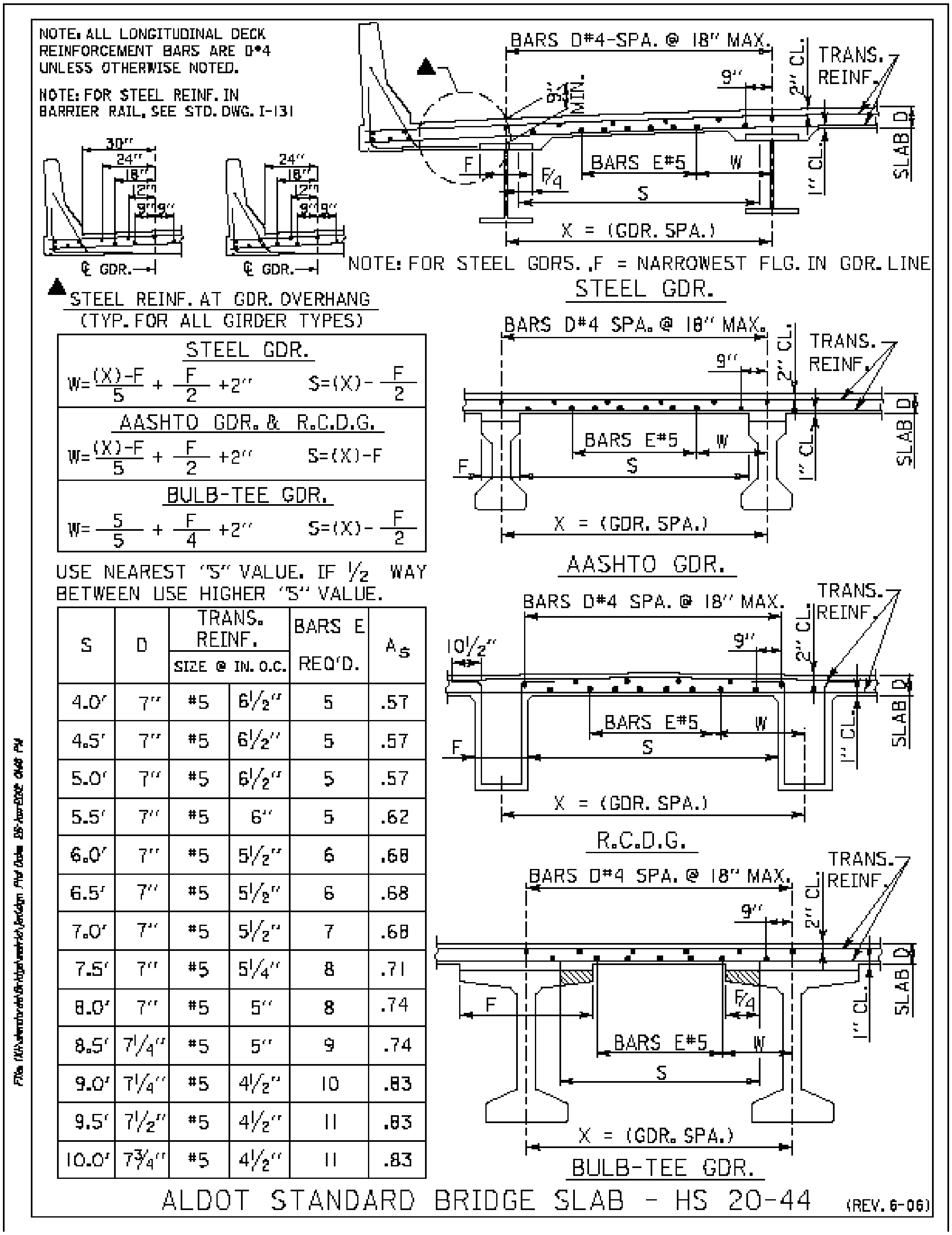
ALDOT design specifications mandate the use of the current edition of "AASHTO Standard Specifications for Highway Bridges" under the HS 20-44 design live load in compliance with the Service Load Design Method (Allowable Stress Design).

To ensure safety and uniformity in bridge design, the State Bridge Engineer provides bridge designers with the ALDOT Standard Bridge Slab details for reinforced concrete (RC) decks supported by girder type: steel girders, AASHTO girders, RC deck girders (T-beams), and Bulb-Tee girders. This detail is presented in Figure 3-2. Slab thickness and deck reinforcement requirements have been predetermined based on girder type and girder spacing (ALDOT Bridge Bureau 2008). All concrete decks require a 28-day compressive strength, f_c', of 4.0 ksi with reinforcement steel of ASTM A615, Grade 60 billet steel. The typical barrier configuration for non skewed bridges is given in ALDOT's Standard Drawing I-131 (ALDOT Bridge Bureau Standard Drawings, 2012) and is presented in Figure 3-1. This barrier has a 15-in base width and extends the entire bridge span on opposite sides of the deck.



Note: From "Standard Barrier Rail for Non Skewed Bridges" by Alabama Department of Transportation, 2012, *ALDOT Bridge Bureau Standard Drawings*, I-131, Sheet 3 of 8. Copyright 2012 by Alabama Department of Transportation. Reprinted with permission.

Figure 3.1 – ALDOT Barrier Rail for Non Skewed Bridges



S	D	TRANS. REINF.		BARS E REQ'D.	A_s
		SIZE	@ IN. O.C.		
4.0'	7"	#5	6½"	5	.57
4.5'	7"	#5	6½"	5	.57
5.0'	7"	#5	6½"	5	.57
5.5'	7"	#5	6"	5	.62
6.0'	7"	#5	5½"	6	.68
6.5'	7"	#5	5½"	6	.68
7.0'	7"	#5	5½"	7	.68
7.5'	7"	#5	5¼"	8	.71
8.0'	7"	#5	5"	8	.74
8.5'	7¼"	#5	5"	9	.74
9.0'	7¼"	#5	4½"	10	.83
9.5'	7½"	#5	4½"	11	.83
10.0'	7¾"	#5	4½"	11	.83

Note: From "ALDOT Bridge Bureau Structures Design and Detail Manual" by Alabama Department of Transportation, 2008, p. 29. Copyright 2008 by Alabama Department of Transportation. Reprinted with permission.

Figure 3.2 – ALDOT Standard Bridge Slab – HS 20-44 Chart

3.3 Methodology

To check the reinforcement criteria as supplied in the ALDOT Standard Bridge Slab –HS 20-44 design chart, a deck analysis was performed with the aid of SAP 2000 and the strength limit checked under AASHTO LRFD Bridge Design Specifications. The design parameters used are in accordance with RC decks supported by typical AASHTO girders and RC Deck-Girder combinations. A deck-girder combination involves the flanges of the girders acting as part of the deck. Two cases are investigated using the line-girder method of analysis: (1) deck sections consisting of four girders and (2) sections with six girders. The moment results of both cases are approximately equal if not exact with the largest differential being less than half a percent. For this reason, the results of Case-2 will not be shown in this report.

Constants used in the deck models are barrier widths of 15-in, overhang length, and girder stem width. The overhang deck length from centerline of each exterior girder is 3-¾-ft. This length is used per ALDOT deck standards from Figure 3.2. The stem width of each T-beam girder is considered as 12-in. Center-to-center girder spacing varied from 5-ft to 11-ft increasing at ½-ft increments resulting in thirteen cross-section deck models. The depth, D, of the concrete deck varied from 7-in to 7¾-in, increasing as the clear span (S) reaches 8 ½-ft or more. S is defined as the clear span distance between two adjacent girders (e.g. for a girder to girder spacing of 5-ft, the clear spacing would be the girder-to-girder spacing minus the girder stem width or 5-ft minus 12-in resulting in a clear spacing of 4-ft). The cross-section of a typical deck-girder bridge model is presented in Figure 3.3.

To ensure bridge safety, all engineering design specifications are geared toward the general principle of supplying member resistance that is greater than or equal to the force effects caused by applied loads. Load and Resistance Factor Design makes use of statistically determined load and resistance factors to achieve this. The general LRFD equation is:

$$\Phi R_n \geq \sum \eta_i \gamma_i Q_i \qquad \textit{Equation 3-1}$$

Where,

Φ = Resistance factor depending on limit state

R_n = Nominal resistance supplied by member

η_i = Load modification factor dependent on ductility, redundancy, importance

γ_i = Load factor dependent on load type

Q_i = Load/force effect dependent on load type

The deck analysis is checked for the strength limit state with the following factors and variables ([AX.X.X]–refers to specified Article in AASHTO LRFD Specifications):

- Φ = 0.9 (for tension controlled sections) [A5.5.4.2.1]
- $R_n = M_n$ = Nominal Moment capacity of ALDOT deck slabs
- $\eta_i = \eta_D \times \eta_R \times \eta_I = 1.0$ [A1.3.3–A1.3.5]
- $\gamma_i = \gamma_{LL} = 1.75$; $\gamma_P = 1.25$ (max) & 0.9 (min) [Table A3.4.1-2]
    - γ_{LL} – Live Load factor
    - γ_P - Permanent Load factor
- $Q_i = M_{LL}$ and M_{DC}
    - M_{LL} - Maximum Moment of Live Load
    - M_{DC} - Dead Load Moment of slab and barrier at $M_{LL\text{-}Max}$ location

To account for dynamic load effects of moving vehicles over decks and deck components, LRFD imposes a 33% impact factor that is added to the static load effect when designing for the strength limit state. This empirical factor represents the dynamic allowance applied to the static wheel loads when vehicles are in motion [A3.6.2]. The load effect is given by the following:

$$Q_{LL+IM} = Q_{LL}(1 + IM) = 1.33Q_{LL} \qquad \textit{Equation 3-2}$$

Where,

Q_{LL+I} = Static live load effect plus the dynamic load allowance

Q_{LL} = Live load force effect

IM = Impact factor of 33%

3.4 Loadings

The greatest live load/force that the bridge decks must withstand comes from the rear tri-axle group of the 97-S truck configuration. This tri-axle is a combination of three 17-kip axles with the center axle separated by 4-ft from the two adjacent axles (See Figure 2.1). In a transverse view of the bridge deck, this live load is comprised of two concentrated loads separated by the 6-ft axle width with each load being 25 ½ -kip, totaling the 51-kip load of the tri-axle. Using the linear multi-step static load generator in SAP 2000 the 51-kip axle is applied to each bridge cross section. The initial placement of the left wheel load is located on the deck overhang 1.5-ft from the centerline of the exterior girder. Six feet across the deck marks the initial position of the right 25.5-kip load. The multi-step generator will move this axle loading along the transverse length of the deck in increments of 1-in per second, creating a moment envelope of the live loads. A typical cross section with the live loading is shown in Figure 3.3 below.

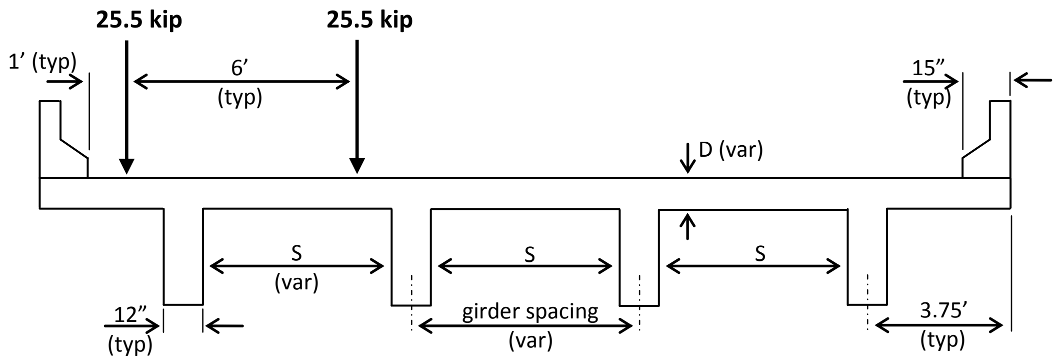


Figure 3.3 – Typical Bridge Cross-Section

The dead loads acting on the deck are the uniform load of the concrete (ksf) and the two point loads from the ALDOT standard barriers (kip/ft). The barrier loads are positioned according to their center of gravity, 5-in from the free edge of each overhang. Using AASHTO's approximate strip method [A4.6.2.1] with the transverse vehicle load placements no closer than 1-ft to the barrier curb face [A3.6.1.3.1], the maximum force effects are determined for each bridge section. The dead loads used for the thirteen deck models are summarized in Table 3-1 below. Loads from future wearing surfaces are not included in the deck analysis per ALDOT design standards.

3.5 SAP 2000 Analysis

After all thirteen deck models, with four girders each, are analyzed in SAP 2000 the maximum positive and negative moments are extracted from the results. Since each deck consisted of four girders, they are treated as a continuous span having both positive and negative moment effects. The maximum moment locations are dependent upon the live load positioning. The locations of maximum positive moments varied slightly depending on clear span but are around 35%-45% of the exterior span length with respect to the exterior support. Negative moment locations are a bit more tedious. For continuous beams, the maximum negative moment occurs at either the exterior support/overhang or the first interior support/girder. Per Article 4.6.2.1.6, maximum moments can be decreased to the respective values at the support face when reinforcement is being selected. With the base of girders modeled at 12-in, the negative moments are taken at locations 6-in from the girder centerline. Each face of the exterior and interior supports had to be checked because the maximum values are dependent on their respective strip widths.

Table 3-1: Dead Load by Clear Spacing & Slab Depth

S	D	$\rho_{concrete}$	w_{slab} *	$A_{barrier}$	$P_{barrier}$ **
ft	in	kcf	ksf	in^2	kip/ft
4	7	0.150	0.0875	293	0.305
4.5	7	0.150	0.0875	293	0.305
5	7	0.150	0.0875	293	0.305
5.5	7	0.150	0.0875	293	0.305
6	7	0.150	0.0875	293	0.305
6.5	7	0.150	0.0875	293	0.305
7	7	0.150	0.0875	293	0.305
7.5	7	0.150	0.0875	293	0.305
8	7	0.150	0.0875	293	0.305
8.5	7 ¼	0.150	0.0906	293	0.305
9	7 ¼	0.150	0.0906	293	0.305
9.5	7 ½	0.150	0.0938	293	0.305
10	7 ¾	0.150	0.0969	293	0.305

* distributed load throughout full length of cross-section

** concentrated load positioned 5" from free end of each overhang

One-way action from a single axle is assumed to determine the transverse live load moment effect. But these moments are not concentrated like one-way analysis calculates. Actually the three axles, with an overall longitudinal spacing of 8-ft, are used to apply the loads, so it becomes imperative to distribute these moments longitudinally. In order to convert these force effects to moments per linear foot in the longitudinal direction, the AASHTO approximate strip method is used [A4.6.2.1]. This technique requires equivalent longitudinal strip widths to be calculated for each of the overhang, the positive, and the negative moment regions. These strips represent the distribution of the force effect caused by the wheel loads throughout the longitudinal direction of the deck. The overhang strip width is a function of the transverse distance between the wheel load on the overhang and the center of the exterior support. Both the positive and negative moment strip widths rely on the center-to-center girder spacing.

Overhang: $SW^{Overhang} = 45 + 10.0X$ *Equation 3 – 3*

$M^{Positive}$: $SW^{+} = 26.0 + 6.6S_G$ *Equation 3 – 4*

$M^{Negative}$: $SW^{-} = 48.0 + 3.0S_G$ *Equation 3 – 5*

Where,

S_G = center-to-center girder spacing (ft)
X = distance of wheel load to centerline of exterior support (ft)
Note: strip width equations return values in inches

The AASHTO LRFD Bridge Design Specifications use multiple presence factors, *m*, for adequately adjusting force effects due to multiple heavy vehicles being present in adjacent lanes [A3.6.1.1.2]. This factor is based on probability of occurrence so the conservative design uses a factor of 1.2 for the presence of a single truck. The LRFD design moment effects due to the 51-kip tri-axle are determined by dividing the product of the maximum moment generated from SAP 2000, $M_{LL\text{-}SAP}$, and the multiple presence factor of 1.2 by the calculated strip width. These positive and negative moments denoted "$M_{LL\text{-}max}$", are displayed in Table 3-2 and Table 3-3. The locations along the deck that these moments act are listed according to the nomenclature used by Barker and Puckett

(2007). Each deck model has three equal spans between the four supports and two cantilevered overhangs totaling five beam-like members. In chronological order, members (1) and (5) are overhangs, members (2) and (4) - exterior spans, and member (3) is the interior span. The first numeral after the "M" represents which member the maximum moment is located. The numeral directly preceding the decimal, and all numerals following, signifies the precise location the moment acts, represented as a percentage of the member's span length. For the deck with a 5-ft girder spacing in Table 3-2, $M_{204.5}$ corresponds to the moment effect acting on the exterior span [member (2)] at a location 45% of the girder spacing (5-ft) measured from the shared support of the previous member, member (1). $M_{204.5}$ occurs on member (2) located 2.25' from the exterior support.

Table 3-2: Maximum Positive Moment from Live Loads

S/S_G* ft	$+M_{LL}$ Location	m	$+M_{LL\text{-}SAP}$ kip-ft	SW** ft	$+M_{LL}$ kip-ft/ft
4/5	$M_{204.50}$	1.2	23.50	4.92	5.74
4.5/5.5	$M_{204.09}$	1.2	25.06	5.19	5.79
5/6	$M_{203.75}$	1.2	26.95	5.47	5.92
5.5/6.5	$M_{203.85}$	1.2	29.32	5.74	6.13
6/7	$M_{203.57}$	1.2	31.94	6.02	6.37
6.5/7.5	$M_{203.67}$	1.2	34.94	6.29	6.66
7/8	$M_{203.44}$	1.2	38.11	6.57	6.96
7.5/8.5	$M_{203.53}$	1.2	41.61	6.84	7.30
8/9	$M_{203.61}$	1.2	45.25	7.12	7.63
8.5/9.5	$M_{203.42}$	1.2	49.15	7.39	7.98
9/10	$M_{203.50}$	1.2	53.23	7.67	8.33
9.5/10.5	$M_{203.57}$	1.2	57.39	7.94	8.67
10/11	$M_{203.41}$	1.2	61.67	8.22	9.01

* Clear Span and Girder Spacing

** Positive moment strip width per A4.6.2.1

Table 3-3: Maximum Negative Moment from Live Loads

S/S_G * ft	$-M_{LL}$ Location	m	$-M_{LL\text{-}SAP}$ kip-ft	SW ** ft	$-M_{LL}$ kip-ft/ft
4/5	$M_{201.00}$	1.2	-32.80	5.00	-7.87
4.5/5.5	$M_{200.91}$	1.2	-32.43	5.00	-7.78
5/6	$M_{200.83}$	1.2	-32.06	5.00	-7.69
5.5/6.5	$M_{200.77}$	1.2	-31.71	5.00	-7.61
6/7	$M_{200.71}$	1.2	-31.38	5.00	-7.53
6.5/7.5	$M_{200.67}$	1.2	-31.07	5.00	-7.46
7/8	$M_{200.63}$	1.2	-30.78	5.00	-7.39
7.5/8.5	$M_{200.59}$	1.2	-30.52	5.00	-7.32
8/9	$M_{200.56}$	1.2	-30.28	5.00	-7.27
8.5/9.5	$M_{200.53}$	1.2	-30.06	5.00	-7.21
9/10	$M_{200.50}$	1.2	-29.85	5.00	-7.16
9.5/10.5	$M_{200.48}$	1.2	-29.66	5.00	-7.12
10/11	$M_{200.45}$	1.2	-29.48	5.00	-7.08

* Clear Span and Girder Spacing

** Overhang moment strip width per A4.6.2.1

Realizing the live loads govern the maximum moment location for each deck model, moments from the dead loads are determined at the corresponding locations. These dead load moments, M_{DC}, which include the effects from the slab and barrier are shown in Tables 3-4 & 3-5.

Once all extreme moment effects from the live and dead loads are determined, the ultimate factored force effect is calculated per the right side of *Equation 3-1*. The specific equation for the ultimate factored moment, M_u, is:

$$M_u = \sum \eta_i \gamma_i M_i = 1.0\gamma_p M_{DC} + 1.0\gamma_{LL} 1.33\, M_{LL} \qquad \textit{Equation 3-6}$$

The permanent load factors are assigned as to produce the extreme force effect. If the additive force effects from the dead loads do achieve this, the permanent load factor, γ_p, is taken as the maximum factor (1.25). However, if the additive values lessen the total force effect, the minimum load factor shall be used (0.9) [A3.4.1]. The ultimate factored force effects and corresponding load factors are given in Tables 3-6 and 3-7.

Table 3-4: Moment from Dead Loads at $+M_{LL}$ Location

S/S_G* ft	$+M_{LL}$ Location	M_{Slab} kip-ft/ft	$M_{Barrier}$ kip-ft/ft	M_{DC} kip-ft/ft
4/5	$M_{204.50}$	-0.1208	-0.4904	-0.6112
4.5/5.5	$M_{204.09}$	-0.1084	-0.5342	-0.6426
5/6	$M_{203.75}$	-0.092	-0.5717	-0.6637
5.5/6.5	$M_{203.85}$	-0.0395	-0.5583	-0.5978
6/7	$M_{203.57}$	-0.0148	-0.5894	-0.6042
6.5/7.5	$M_{203.67}$	0.0449	-0.5768	-0.5319
7/8	$M_{203.44}$	0.0770	-0.603	-0.5260
7.5/8.5	$M_{203.53}$	0.1439	-0.5912	-0.4473
8/9	$M_{203.61}$	0.2131	-0.5808	-0.3677
8.5/9.5	$M_{203.42}$	0.2655	-0.603	-0.3375
9/10	$M_{203.50}$	0.3449	-0.593	-0.2481
9.5/10.5	$M_{203.57}$	0.4421	-0.5841	-0.1420
10/11	$M_{203.41}$	0.5165	-0.6031	-0.0866

* Clear Span and Girder Spacing

Table 3-5: Moment from Dead Loads at $-M_{LL}$ Location

S/S_G* ft	$-M_{LL}$ Location	M_{Slab} kip-ft/ft	$M_{Barrier}$ kip-ft/ft	M_{DC} kip-ft/ft
4/5	$M_{201.00}$	-0.4672	-0.8983	-1.3655
4.5/5.5	$M_{200.91}$	-0.4644	-0.908	-1.3724
5/6	$M_{200.83}$	-0.4608	-0.9164	-1.3772
5.5/6.5	$M_{200.77}$	-0.4565	-0.9235	-1.3800
6/7	$M_{200.71}$	-0.4515	-0.9297	-1.3812
6.5/7.5	$M_{200.67}$	-0.4461	-0.9351	-1.3812
7/8	$M_{200.63}$	-0.4403	-0.9399	-1.3802
7.5/8.5	$M_{200.59}$	-0.4342	-0.9442	-1.3784
8/9	$M_{200.56}$	-0.4278	-0.9480	-1.3758
8.5/9.5	$M_{200.53}$	-0.4360	-0.9514	-1.3874
9/10	$M_{200.50}$	-0.4289	-0.9545	-1.3834
9.5/10.5	$M_{200.48}$	-0.4366	-0.9573	-1.3939
10/11	$M_{200.45}$	-0.4430	-0.9599	-1.4029

* Clear Span and Girder Spacing

Table 3-6: Positive Ultimate Factored Design Moment

S/S_G ft	γ_p	M_{DC} k-ft/ft	γ_{LL}	(1+IM)	+ M_{LL} k-ft/ft	+ M_u k-ft/ft
4/5	1.25	-0.611	1.75	1.33	5.74	12.59
4.5/5.5	1.25	-0.643	1.75	1.33	5.79	12.68
5/6	1.25	-0.664	1.75	1.33	5.92	12.94
5.5/6.5	1.25	-0.598	1.75	1.33	6.13	13.52
6/7	1.25	-0.604	1.75	1.33	6.37	14.07
6.5/7.5	0.9	-0.532	1.75	1.33	6.66	15.03
7/8	0.9	-0.526	1.75	1.33	6.96	15.74
7.5/8.5	0.9	-0.447	1.75	1.33	7.30	16.58
8/9	0.9	-0.368	1.75	1.33	7.63	17.43
8.5/9.5	0.9	-0.338	1.75	1.33	7.98	18.27
9/10	0.9	-0.248	1.75	1.33	8.33	19.17
9.5/10.5	0.9	-0.142	1.75	1.33	8.67	20.06
10/11	0.9	-0.087	1.75	1.33	9.01	20.88

Table 3-7: Negative Ultimate Factored Design Moment

S/S_G ft	γ_p	M_{DC} k-ft/ft	γ_{LL}	(1 + IM)	- M_{LL} k-ft/ft	- M_u k-ft/ft
4/5	1.25	-1.37	1.75	1.33	-7.87	-20.03
4.5/5.5	1.25	-1.37	1.75	1.33	-7.78	-19.83
5/6	1.25	-1.38	1.75	1.33	-7.69	-19.63
5.5/6.5	1.25	-1.38	1.75	1.33	-7.61	-19.44
6/7	1.25	-1.38	1.75	1.33	-7.53	-19.26
6.5/7.5	1.25	-1.38	1.75	1.33	-7.46	-19.08
7/8	1.25	-1.38	1.75	1.33	-7.39	-18.92
7.5/8.5	1.25	-1.38	1.75	1.33	-7.33	-18.77
8/9	1.25	-1.38	1.75	1.33	-7.27	-18.63
8.5/9.5	1.25	-1.39	1.75	1.33	-7.21	-18.53
9/10	1.25	-1.38	1.75	1.33	-7.16	-18.40
9.5/10.5	1.25	-1.394	1.75	1.33	-7.12	-18.31
10/11	1.25	-1.40	1.75	1.33	-7.08	-18.22

3.6 ALDOT Standard Bridge Slab Reinforcement Evaluation

Using the LRFD maximum moment effects established in Section 3.5, the transverse deck reinforcement provided in the ALDOT Standard Bridge Slab chart (Figure 3.1) is evaluated using LRFD methods at the strength limit state. This task is achieved by checking both the positive and negative moment reinforcement for adequate ductility and tensile strength. Several key assumptions are made when designing RC members for flexural and axial force effects at the this limit state [A5.7.2.1].

- Member capacity is based on compatibility and equilibrium (Figure 3.4)
- Plane sections before bending remain plane once bending has taken place
- Reinforcement strain is directly proportional to the distance to the neutral axis
- Maximum compressive strain of concrete is 0.003 at extreme compressive fiber
- Tensile strength of concrete is insignificant and not included in design
- Balanced strain is achieved when steel yields as the concrete strain reaches its maximum of 0.003-in/in
- A compression-controlled section has concrete strains of 0.003 before steel yields
    - Grade 60 steel – yielding strain, $\varepsilon_t = 0.002$
    - Brittle failure
- A tension-controlled section has tensile strains, ε_t, ≥ 0.005 at max concrete strain
    - Large deformations and cracking is expected before failure

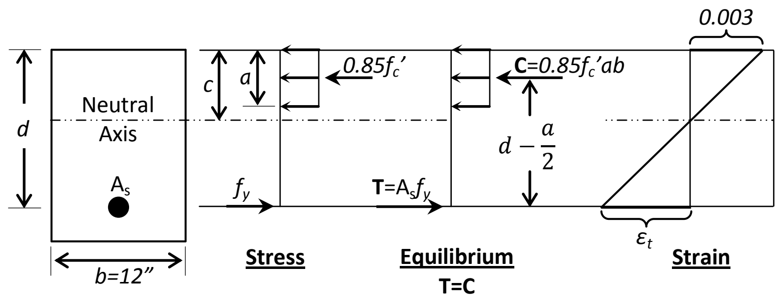


Figure 3.4 – Stress-Strain Diagram of Typical RC Section

Where,

d – Effective depth of extreme compression fiber to centroid of tensile reinforcement (in)

c – Depth from extreme compression fiber to neutral axis of section (in)

a – Depth of equivalent compression stress block (in)

$$a = \beta_1 c \qquad \textit{Equation 3-7}$$

β_1 – Factor relating depth of equivalent compressive stress block to depth of neutral axis

- for f_c' of 4000-psi → $\beta_1 = 0.85$

b – 1-ft longitudinal width of deck section (in)

f_y – Yield strength of steel (ksi)

f_c' – Compressive strength of concrete (ksi)

ε_t – Net tensile strain of reinforcement (in/in)

Neutral Axis – Separates the tension and compression zones of a section

3.6.1 Positive Reinforcement

The function of the positive reinforcement is to provide tensile strength against the flexural phenomena of positive moments. The location of this steel is in the tension region of the deck which is below the neutral axis when positive moment is present. In order to carry out the desired strength checks, the material properties of the deck must be known. Table 3-8 lists the required slab properties as described in the ALDOT Bridge Bureau Structures Design and Detailing Manual.

Table 3-8: ALDOT Bridge Slab Properties

CONCRETE	REINFORCEMENT
Normal Weight	ASTM A615, Gr 60 Billet Steel
f_c' - 4000 psi	f_y - 60 ksi
<u>Clear Cover:</u>	Bar Size - #5
Top & Side - 2"	Dia. bar, d_b - 0.625"
Bottom - 1"	Area steel, A_s - 0.31 in^2

Resistance Factors – [A5.5.4.2.1]

If too much steel is present in deck slabs it can have adverse affects on safety that can lead to brittle failure. Prior to 2005 a maximum allowable reinforcement check was used to verify ductility in flexural members [A5.7.3.3.1]. Ductile members will experience large visible deformations and cracking before failure occurs. This reinforcement limitation is based on the ratio of the compression zone depth to the total compression and tension zone depth in a section, *c/d*. The maximum reinforcement check has since been altered and is now governed by the resistance factor, Φ, which varies according to the reinforcement strain type: tension-controlled, compression-controlled, or in a transitional state between the two. Table 3-9 shows the resistance factors for use at the strength limit state.

Table 3-9: Resistance Factors

ε_t	Φ
Tension-controlled $\varepsilon_t \geq 0.005$	0.9
Transitional $0.002 < \varepsilon_t < 0.005$	$0.65+0.15(d/c-1)$
Compression-controlled $\varepsilon_t \leq 0.002$	0.75

From Figure 3.4, the net tensile strain is explained as

$$\varepsilon_t = \frac{(d-c)}{c}0.003 \qquad \textit{Equation 3-8}$$

To determine the location of the neutral axis from the extreme compression fiber, *c*, the equivalent compressive stress block depth, *a*, is determined and input into *Equation 3-7*.

$$a = \frac{A_s f_y}{0.85 f_c' b}$$ *Equation 3-9*

Utilizing *Equations 3-7* thru *3*-9 with the material properties specified in Table 3-9, the appropriate resistance factor for the nominal moment capacity is determined. Table 3-10 lists the results.

Table 3-10: Resistance Factors for Positive Reinforcement

S ft	D in	d_{pos} in	c in	ε_t in/in	> 0.005	Φ
4.0	7	5.69	0.986	0.014	0.005	0.9
4.5	7	5.69	0.986	0.014	0.005	0.9
5.0	7	5.69	0.986	0.014	0.005	0.9
5.5	7	5.69	1.073	0.013	0.005	0.9
6.0	7	5.69	1.176	0.012	0.005	0.9
6.5	7	5.69	1.176	0.012	0.005	0.9
7.0	7	5.69	1.176	0.012	0.005	0.9
7.5	7	5.69	1.228	0.011	0.005	0.9
8.0	7	5.69	1.280	0.010	0.005	0.9
8.5	7 ¼	5.94	1.280	0.011	0.005	0.9
9.0	7 ¼	5.94	1.436	0.009	0.005	0.9
9.5	7 ½	6.19	1.436	0.010	0.005	0.9
10.0	7 ¾	6.44	1.436	0.010	0.005	0.9

The net tensile strain in the thirteen deck members reveals that each section is tension-controlled as this primary reinforcement yields to a point that expresses ductile behavior. In other words, large deflections and cracking will occur before the deck fails.

Minimum Reinforcement – [A5.7.3.3.2]

The minimum reinforcement limitation for components in flexure requires enough reinforcement be present so that the factored nominal strength, $\Phi M_n = M_u$, is no smaller than the minimum value of:

(1) $1.2M_{cr}$

(2) $1.33M_u$

M_{cr} – Cracking moment of concrete

$$M_{cr} = S_{nc} f_r \qquad \textit{Equation 3-10}$$

S_{nc} – Section modulus for the extreme fiber of the noncomposite section where tensile stress is caused by external loads

$$S_{nc} = \frac{bh^2}{6} \quad (\text{in}^3) \qquad \textit{Equation 3-11}$$

b – Longitudinal base length = 12-in

h – Height of slab/Depth

f_r – Rupture modulus of concrete (ksi) [A5.4.2.6]

$$f_r = 0.37\sqrt{f_c'} \quad (\text{ksi}) \qquad \text{[for normal concrete]} \qquad \textit{Equation 3-12}$$

$$\Phi M_n = \Phi A_s f_y (d - \frac{a}{2}) \qquad \textit{Equation 3-13}$$

The LRFD check of tensile strength for the ALDOT specified reinforcement is listed in Table 3-11.

The positive reinforcement supplied by ALDOT contains acceptable strength for the 51-kip tri-axle of the 97-S for all clear spacing except from 7.5-ft – 8.5-ft. However, the factored moment capacities that do not meet the design moment effects are less than 2%. In this case, the reinforcement selection may be deemed as sufficient at the engineer's discretion. To demonstrate how redesign takes place, new reinforcement is selected. Using *Equation 3-14*, trial areas of steel are selected for this group. After the ductility and strength requirements are checked, an appropriate reinforcement is recommended which is shown in Table 3-12.

$$trial - A_s = \frac{M_u}{4d} \qquad \textit{Equation 3-14}$$

Table 3-11: Minimum Positive Reinforcement – [A5.7.3.3.2]

S	D	S_{nc}	f_r	$1.2M_{cr}$	$1.33M_u$	$+ M_u$	≤	ΦM_n	CHECK
ft	in	in^3	ksi	k-ft/ft	k-ft/ft	k-ft/ft		k-ft/ft	
4.0	7	98	0.74	7.25	16.74	12.59	≤	13.51	✓
4.5	7	98	0.74	7.25	16.86	12.68	≤	13.51	✓
5.0	7	98	0.74	7.25	17.21	12.94	≤	13.51	✓
5.5	7	98	0.74	7.25	17.98	13.52	≤	14.60	✓
6.0	7	98	0.74	7.25	18.72	14.07	≤	15.87	✓
6.5	7	98	0.74	7.25	19.99	15.03	≤	15.87	✓
7.0	7	98	0.74	7.25	20.93	15.74	≤	15.87	✓
7.5	7	98	0.74	7.25	22.06	16.58	>	16.50	✘
8.0	7	98	0.74	7.25	23.18	17.43	>	17.13	✘
8.5	7 ¼	105	0.74	7.78	24.30	18.27	>	17.96	✘
9.0	7 ¼	105	0.74	7.78	25.49	19.17	≤	19.90	✓
9.5	7 ½	113	0.74	8.33	26.67	20.06	≤	20.83	✓
10	7 ¾	120	0.74	8.89	27.78	20.88	≤	21.76	✓

Table 3-12: Recommended Positive Reinforcement

S	$+ M_u$	*trial–A_s*	$A_{s\text{-sup}}$	Resistance Factor				Strength Requirement			
ft	k-ft/ft	in^2/ft	in^2/ft	*c*	*d/c*	ε_t	Φ	+ Mu	≤	ΦM_n	Check
7.5	16.58	0.73	0.74	1.28	4.44	0.0081	0.9	16.58	≤	17.13	✓
8.0	17.43	0.77	0.82	1.42	4.01	0.0064	0.9	17.43	≤	18.76	✓
8.5	18.27	0.77	0.82	1.42	4.19	0.0067	0.9	18.27	≤	19.68	✓

Recommended Reinforcement

S	D	TRANS. REINF		A_s
ft	in	Size @ in. o.c.		in^2/ft
7.5	7	#5	5	0.74
8.0	7	#5	4.5	0.82
8.5	7 ¼	#5	4.5	0.82

3.6.2 Negative Reinforcement

The usefulness of negative reinforcement is to provide strength and ductility in the same manner as the positive reinforcement. The difference between the two types stems from the placement of the negative reinforcement. Since negative bending moment causes tension in the upper region of a typical section, the location of transverse reinforcement is in the top portion of the slab with enough clear cover to be considered fully bonded with the concrete.

Resistance Factors – [A5.7.3.3.1]

The amount of primary reinforcement suggested by ALDOT applies to both positive and negative moment regions for a specified clear spacing (see Figure 3.5). Therefore, the depth of the neutral axis from the extreme compressive fiber, c, remains the same as calculated in Table 3-10. However, the tensile strain in the reinforcement of *Equation 3-7* will change because the effective depth, d_{neg}, is altered due to the top cover requirements differing from the bottom requirements. Table 3-13 shows the resistance factors required for the maximum limit of negative reinforcement. The reinforcement for members experiencing negative moment is adequate for all clear spans checked as each section is tension-controlled. Therefore sufficient ductility exists in all members.

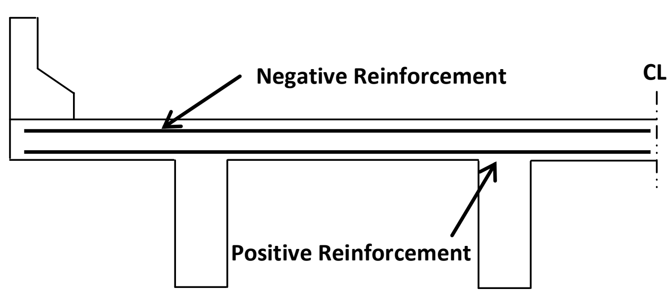


Figure 3.5 – Primary Reinforcement

Table 3-13: Resistance Factors for Negative Reinforcement

S ft	D in	d_{neg} in	c in	ε_t in/in	> 0.005	Φ
4.0	7	4.69	0.986	0.011	0.005	0.9
4.5	7	4.69	0.986	0.011	0.005	0.9
5.0	7	4.69	0.986	0.011	0.005	0.9
5.5	7	4.69	1.073	0.010	0.005	0.9
6.0	7	4.69	1.176	0.009	0.005	0.9
6.5	7	4.69	1.176	0.009	0.005	0.9
7.0	7	4.69	1.176	0.009	0.005	0.9
7.5	7	4.69	1.228	0.008	0.005	0.9
8.0	7	4.69	1.280	0.008	0.005	0.9
8.5	7 ¼	4.94	1.280	0.009	0.005	0.9
9.0	7 ¼	4.94	1.436	0.007	0.005	0.9
9.5	7 ½	5.19	1.436	0.008	0.005	0.9
10.0	7 ¾	5.44	1.436	0.008	0.005	0.9

Minimum Reinforcement – [A5.7.3.3.2]

The strength requirement for negative reinforcement is checked in the same manner as the positive reinforcement. The design check is listed in Table 3-14. The negative reinforcement given in the ALDOT Standard Bridge Slab chart does not meet the strength requirements of the LRFD specifications. The extreme cases occur as clear span lengths are shortened. This is due to the maximum force effects experienced in the overhang. Further design analysis is conducted with the forethought of establishing recommendations for negative reinforcement that meets strength requirements. Trial steel areas are selected once again with the use of *Equation 3-14*, and both maximum and minimum reinforcement limitations are examined. The results are listed in Table 3-15.

Table 3-14: Minimum Negative Reinforcement – [A5.7.3.3.2]

S	D	S_{nc}	f_r	$1.2M_{cr}$	$1.33M_u$	$-M_u$	$\leq$	ΦM_n	CHECK
ft	in	in^3	ksi	k-ft/ft	k-ft/ft	k-ft/ft		k-ft/ft	
4.0	7	98	0.74	7.25	26.64	20.03	>	10.9	✘
4.5	7	98	0.74	7.25	26.38	19.83	>	10.9	✘
5.0	7	98	0.74	7.25	26.11	19.63	>	10.9	✘
5.5	7	98	0.74	7.25	25.85	19.44	>	11.8	✘
6.0	7	98	0.74	7.25	25.61	19.26	>	12.8	✘
6.5	7	98	0.74	7.25	25.38	19.08	>	12.8	✘
7.0	7	98	0.74	7.25	25.16	18.92	>	12.8	✘
7.5	7	98	0.74	7.25	24.97	18.77	>	13.3	✘
8.0	7	98	0.74	7.25	24.78	18.63	>	13.8	✘
8.5	7 ¼	105	0.74	7.78	24.64	18.53	>	14.6	✘
9.0	7 ¼	105	0.74	7.78	24.48	18.40	>	16.2	✘
9.5	7 ½	113	0.74	8.33	24.35	18.31	>	17.1	✘
10.	7 ¾	120	0.74	8.89	24.23	18.22	>	18.0	✘

For 7-in deck slabs, the amount of steel added to supply sufficient resistance will decrease the ductility of the member shown by the lower resistance factor values in Table 3-15. It is suggested that slab depths be increased accordingly to maintain ductility in the members. However, the reinforcement for clear spans of 8.5-ft to 10-ft do meet LRFD Specifications. All deck reinforcement designs shown are done using the conservative approach of considering each section as being singly reinforced as compared to the actual double reinforcement provided. When checked, the doubly reinforced sections added very little additional strength, less than a 2% increase. Sections that do exceed strength requirements require parallel reinforcement placement very close to each other. A cost-benefit analysis is recommended to determine the most viable option for increasing strength: (1) thicken slabs by adding concrete, (2) increasing the amount of the #5 reinforcement rods used per foot, or (3) increasing the reinforcement area above #5 bars.

Table 3-15: Design Analysis of Negative Reinforcement

S	- M_u	*trial-* A_S	$A_{s\text{-}sup}$	Resistance Factor				Strength Requirement			
ft	k-ft/ft	in²/ft	in²/ft	c	d/c	ε_t	Φ	- Mu	≤	ΦM_n	Check
4.0	20.03	1.07	1.23	2.13	2.20	0.0036	0.83	20.03	>	19.32	✗
4.5	19.83	1.06	1.23	2.13	2.20	0.0036	0.83	19.83	>	19.32	✗
5.0	19.63	1.05	1.05	1.82	2.58	0.0047	0.89	19.63	>	18.23	✗
5.5	19.44	1.04	1.05	1.82	2.58	0.0047	0.89	19.44	>	18.23	✗
6.0	19.26	1.03	1.05	1.82	2.58	0.0047	0.89	19.26	>	18.23	✗
6.5	19.08	1.02	1.05	1.82	2.58	0.0047	0.89	19.08	>	18.23	✗
7.0	18.92	1.01	1.05	1.82	2.58	0.0047	0.89	18.92	>	18.23	✗
7.5	18.77	1.00	1.05	1.82	2.58	0.0047	0.89	18.77	>	18.23	✗
8.0	18.63	0.99	1.05	1.82	2.58	0.0047	0.89	18.63	>	18.23	✗
8.5	18.53	0.94	1.05	1.82	2.72	0.0052	0.9	18.53	≤	19.68	✓
9.0	18.40	0.93	1.05	1.82	2.72	0.0052	0.9	18.40	≤	19.68	✓
9.5	18.31	0.88	0.92	1.59	3.26	0.0068	0.9	18.31	≤	18.68	✓
10	18.22	0.84	0.92	1.59	3.42	0.0072	0.9	18.22	≤	19.71	✓

Recommended Reinforcement

S	D	TRANS REINF		A_s
ft	in	Size	@ in. o.c.	in²/ft
8.5	7 ¼	#5	3.5	1.05
9.0	7 ¼	#5	3.5	1.05
9.5	7 ½	#5	4	0.92
10	7 ¾	#5	4	0.92

The results presented in Chapter 3 should not be taken as absolute but rather an informative measuring tool for comparative purposes. Uncertainties are present due to the live load force effects determined by the approximate strip method. The strip width equations are not intended to model the effective widths of the 51-kip tri-axle but specifically the 32-kip axle load of the HS20-44 design truck. A three dimensional finite element analysis is recommended for producing accurate force effects caused by the 97-kip vehicles. This model type will generate results based on detailed influence surfaces rather than the approximate methods used in this report.

CHAPTER 4 SUMMARY, CONCLUSIONS, & RECOMMENDATIONS

4.1 Summary

If the US House of Representatives passes H.R. 763 and states opt to allow six-axle 97-kip trucks to operate on highways, all bridge types will see an increase in shear and moment. To quantify these increases, the UAB School of Civil Engineering conducted a general study of the force effects that heavier trucks have on simple and continuous span bridges. The two proposed six-axle trucks have front-to-rear axle spacing of 40-ft (97-S) and 65-ft (97-TRB). Using linear static line-girder models combined with structural analysis software, the force effects caused by the proposed trucks are compared to those generated by the loadings of three base models: AASHTO Standard Specifications, AASHTO LRFD Specifications, and the loadings of five potentially critical Alabama legal loads. By calculating the shear and moment ratios of the proposed 97-kip truck models to each base model, the increased force effects are revealed.

The effect that the proposed heavier vehicles have on bridge decks is also compared to the standard bridge slab design issued by ALDOT Bridge Bureau. Bending moments govern the critical effect in slab design. For this reason, the live and dead load moments are determined by using the proposed vehicular load models in coordination with the concrete dead loads used in standard ALDOT bridge decks. The factored member capacity of each RC section supplied by ALDOT was checked against the factored ultimate moment using LRFD and line-girder techniques. Sections that did not meet adequate requirements at the strength limit state were redesigned with increased reinforcement. The conclusions and recommendations collected from this study are listed below.

4.2 Conclusions

Simple Spans – minimum span length of 20-ft & maximum of 300-ft

- The 97-S induces greater shear and moment effects than the longer 97-TRB. Axle spacing is a critical factor when determining force effects on bridges. For trucks of the same GVW, decreasing axle spacing will increase the magnitude of the force effect.
- Maximum force effects (i.e. shear, moment, or both) from the 97-S truck exceed the effects produced by the design loadings from the AASHTO Standard Specifications by over 20% on simple span bridges from 80-ft to 155-ft in length. Maximum force effects (i.e. shear, moment, or both) from the 97-TRB truck exceed the effects produced by the design loadings from the AASHTO Standard Specifications by over 5% on simple span bridges from 110-ft to 150-ft in length but the greatest effect (shear) increase is not more than 13%.
- The critical shear and moment developed from the design loads of the AASHTO LRFD Specifications completely envelope the force effects from both 97-kip trucks
- Compared to the five Alabama Legal loads investigated, the 97-TRB initiates greater effects at spans of 100-ft or longer while the 97-S produces higher effects on spans of 55-ft and longer.

- HL-93 loadings from the AASHTO LRFD Specifications generate design effects that are 17% to over 70% above the design values resulting from the HS20-44 design loading and the AML from the AASHTO Standard Specifications. This equates to additional safety in LRFD bridge design as well as the potential to accommodate trucks with increased gross vehicle weight.

Continuous Spans – span ratio of 1:1 & max span length of 150-ft

- The 97-S generally produces greater shear and moments than the 97-TRB. The only instance when this is not valid is the negative moment effect for bridge spans of 45-ft to 80-ft in length.
- HL-93 loading provides unfactored design shear that exceed both proposed truck models. The shear effect from the 97-S and 97-TRB is a maximum 86% and 71% of the design shear, respectively.
- All critical moments from HL-93 loading envelope both 97-kip trucks. All of the positive moments from the 97-TRB are exceeded by at least 50% and all negative moments by 22%. The moment effect from the 97-S and 97-TRB is a maximum 96% and 82% of the design moment, respectively.
- The HS20-44 design loading and AML of the Standard Specifications do not adequately represent the critical effects from the proposed 97-kip models. At the end supports, the 97-S causes a shear increase of 20% or greater on span lengths of 90-ft and longer while the 97-TRB induces a 10% increase on spans 130-ft and longer. At the center support of 105-ft continuous spans, the maximum shear increase from the 97-S and 97-TRB is 27% and 13% respectively. The maximum

positive moment created by the 97-S loading results in increased moment values for all spans above 35-ft while the 97-TRB demonstrates increases on spans 130-ft and longer. The maximum negative moment of the 97-S exceeds the moment effect from the design loading by 31% and the 97-TRB exceeds this effect by 41%.

- When considering positive moment effects, the critical load of the Alabama Legal Loads is the 19-ft 75-kip Alabama Tri-Axle. For all continuous spans up to 150-ft, the tri-axle causes greater positive bending moment than the 97-TRB. However, the moment produced from the 97-S exceeds the Alabama Tri-Axle on continuous spans greater than 80x80-ft long.

Transverse Deck

- The positive reinforcement currently supplied in the ALDOT Standard Bridge Slab chart satisfies the LRFD strength requirements for the critical axle grouping of the 97-kip vehicle for all clear spans between girders except for those of 7½-ft to 8½-ft
- The negative reinforcement supplied does not meet LRFD specifications for either 97-kip truck

4.3 Recommendations

Simple and Continuous Spans

- The 97-S produces greater force effects on all simple span and most continuous span bridges than the longer 97-TRB. More analysis is needed to evaluate bridge safety and the associated costs before the 97-S configuration is considered a viable option for 97-kip trucks.

- LRFD methods should be adopted by state agencies because the design force effects from the notional HL-93 loading effectively envelope heavier trucks. GVW increases of 20% or more are expected in the future so bridges must be designed to withstand the greater force effects. The HS20-44 loading under-designs the force effects that heavier vehicles demonstrate.
- In terms of the force effects created, the 97-TRB is the better alternative compared to the 97-S. This should equate to less construction cost required to strengthen or build new bridges. However, the overall cost depends on several complicated factors that have only been reasonably estimated at the global level. Additional research is required in order to justify heavier trucks on the IHS.

Transverse Deck

- ALDOT Bridge Bureau may need to revise the standard bridge designs to accommodate heavier trucks. The current primary reinforcement for various bridge slabs does not supply adequate resistance for the 51-kip tri-axle load in accordance with the AASHTO LRFD Specifications using the approximate strip method.
- Three-dimensional finite element deck models should be used to determine the force effects that heavier trucks induce in bridge slabs. These effects will accurately reflect those from actual field data, and can be used to better assess the need to update standard designs.

4.3.1 Future Research

In order to effectively quantify the effects that increasing truck weight will have on bridges, more extensive and detailed analysis is required. The force effects determined from this sensitivity analysis only consider two hypothetical 97-kip truck configurations with six-axles and a constant axle spacing. Additional vehicles with variable axle spacing and overall length need to be examined to produce optimum configurations of increased GVW while limiting the impacts on the bridge network.

When heavier vehicles are allowed to operate, certain bridges will likely be overstressed. Overstressing bridge elements can result in decreased service life and more rapid accumulation of damage, but the extent of damage will be dependent on bridge type, bridge age, construction method utilized, geographical location, average daily truck traffic, etc. Fatigue damage is not covered in the analysis but plays a role regarding the impacts on bridges and should be considered in future studies.

State agencies should expect an increase in total bridge cost and a standard methodology for evaluating bridges is needed to arrive at this value. All factors listed herein should be included, but it is essential to verify that the increased benefits outweigh additional costs. What percentage of the annual commercial truck traffic will switch to heavier vehicles? Will uniformity in weight limits exist between states? Restructuring the bridge network will be gradual at best but should begin on routes that are expected to house the highest demand for increased GVW, supplying the greatest return on the investment. Once the facts have been collected and sorted, the costs impacts of heavier trucks can be measured.

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Appendix

Additional Shear and Moment Simple Span Data

Simple Span–Maximum Shear due to Vehicular Loads

Span ft	97-S kip	97-TRB kip	Standard kip	LRFD kip	AL Legal kip
155	86.45	79.63	75.60	117.3	74.35
160	86.78	80.17	77.20	119.0	74.65
165	87.08	80.68	78.80	120.7	74.94
170	87.38	81.16	80.40	122.4	75.20
175	87.65	81.61	82.00	124.2	75.45
180	87.91	82.04	83.60	125.9	75.69
185	88.16	82.45	85.20	127.6	75.92
190	88.39	82.83	86.80	129.3	76.13
195	88.61	83.19	88.40	131.0	76.33
200	88.82	83.54	90.00	132.6	76.52
205	89.02	83.87	91.60	134.3	76.70
210	89.21	84.18	93.20	136.0	76.88
215	89.39	84.48	94.80	137.7	77.04
220	89.56	84.76	96.40	139.3	77.20
225	89.73	85.03	98.00	141.0	77.35
230	89.89	85.29	99.60	142.7	77.50
235	90.04	85.54	101.20	144.3	77.64
240	90.18	85.78	102.80	146.0	77.77
245	90.32	86.01	104.40	147.7	77.90
250	90.46	86.23	106.00	149.3	78.02
255	90.58	86.44	107.60	151.0	78.13
260	90.71	86.64	109.20	152.6	78.25
265	90.83	86.84	110.80	154.3	78.36
270	90.94	87.03	112.40	155.9	78.46
275	91.05	87.21	114.00	157.6	78.56
280	91.16	87.38	115.60	159.2	78.66
285	91.26	87.55	117.20	160.8	78.75
290	91.36	87.72	118.80	162.5	78.84
295	91.45	87.87	120.40	164.1	78.93
300	91.55	88.03	122.00	165.8	79.01

Simple Span–Maximum Moment due to Vehicular Loads

Span ft	97-S kip-ft	97-TRB kip-ft	Standard kip-ft	LRFD kip-ft	AL Legal kip-ft
155	3137	2727	2620	4433	2720
160	3258	2847	2768	4649	2813
165	3379	2967	2921	4869	2907
170	3500	3088	3077	5093	3001
175	3621	3208	3238	5320	3097
180	3742	3328	3402	5552	3202
185	3863	3449	3571	5788	3307
190	3984	3569	3743	6028	3411
195	4105	3689	3920	6272	3516
200	4226	3810	4100	6520	3621
205	4347	3930	4285	6772	3725
210	4468	4051	4473	7028	3830
215	4589	4172	4666	7288	3935
220	4710	4292	4862	7552	4040
225	4831	4413	5063	7820	4145
230	4952	4533	5267	8092	4249
235	5073	4654	5476	8368	4354
240	5194	4775	5688	8648	4459
245	5315	4896	5905	8932	4564
250	5436	5016	6125	9220	4669
255	5557	5137	6350	9512	4774
260	5678	5258	6578	9808	4878
265	5799	5379	6811	10108	4983
270	5921	5500	7047	10412	5088
275	6042	5620	7288	10720	5193
280	6163	5741	7532	11032	5298
285	6284	5862	7781	11348	5403
290	6405	5983	8033	11668	5508
295	6526	6104	8290	11992	5612
300	6647	6225	8550	12320	5717